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From the editor . . .

The **PCI JOURNAL** is the second publication to be initiated and put into production by the Institute within the past year.

PC ITEMS, our monthly news bulletin, reports on activities of members and the Institute and presents views of installations, and latest developments in prestressing.

The **JOURNAL** is our technical publication. It will present authoritative papers by outstanding men in design, production and application of prestressed concrete.

We have set high editorial standards for the **JOURNAL.** It will contain a wealth of material on all phases of prestressing. It is dedicated to helping you better understand and participate in this growing industry. The **JOURNAL** is for you—to answer your questions, give you friendly assistance and help you get real satisfaction out of the potential of prestressed concrete.

Write us, won't you, if there are things you'd like to know, subjects you'd like to see presented in the **JOURNAL.** That's what we're for!

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Editorial contributions to the **JOURNAL** are welcomed, but publication cannot be guaranteed. All manuscripts should be submitted in duplicate and will be reviewed and approved by the Technical Editor prior to publication. Advertisements of products, services and materials allied to the prestressed concrete industry are welcomed in the **JOURNAL.** Address all inquiries concerning advertising rates and mechanical specifications to the Publication Office.

What is P.C.I.?

By J. Rex Farrior, Jr.

Legal Counselor for P. C. I.
Tampa, Florida

P.C.I. is legally known as Prestressed Concrete Institute, Inc., a corporation not for profit organized and existing under the statutes of Florida, Chapter 617. The Institute is legally recognized as a scientific, charitable and benevolent association.

The original charter was approved by the Circuit Court at Tampa, Florida, on June 18, 1954. Officially, the object of the Institute is the formation of an association of manufacturers of prestressed concrete and of persons and companies in allied or related industries for the mutual exchange of information and ideas; and generally to do anything beneficial for its members and the prestressing business which does not violate the laws of Florida or the United States.

The charter provides for six classes of members: Active, for prestressed manufacturers; Associate, for related businesses; Professional, for architects and engineers; Junior, for architects and engineers in training; Student, for students enrolled in accredited architectural and engineering colleges, and Honorary, for such persons as the Board of Directors may wish to honor.

From a tax standpoint, the Institute enjoys an enviable position. Being a valid non-profit corporation, it is not required to pay any tax on its receipts, and therefore 100 per cent of its income can be devoted to research and other useful projects.

Fortunately, also, each member's dues and contributions are completely deductible by the member for income tax purposes.

Both tax factors therefore guarantee to the Institute's members more than full value for each dollar invested.

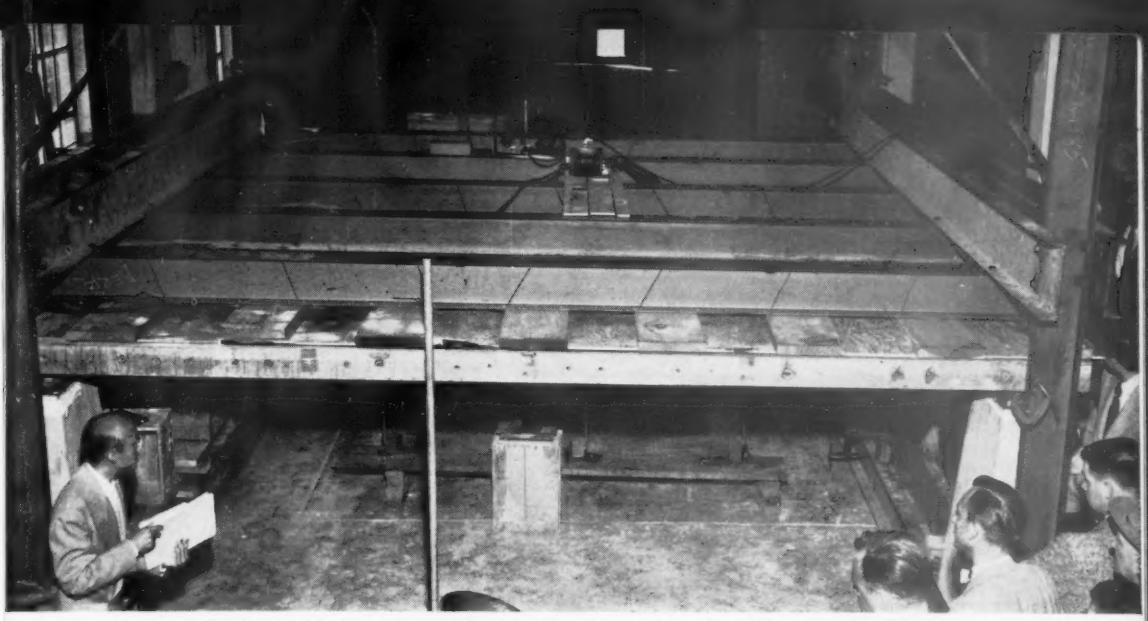


FIG. 2. Testing a 5-inch prestressed concrete slab, uniform load being applied by air pressure in plastic bags (University of California).

CAN WE DESIGN PRESTRESSED CONCRETE BY ALLOWABLE STRESSES

By DR. T. Y. LIN

Associate Professor, Civil Engineering
University of California, Berkeley

(I) INTRODUCTION

Prestressed-concrete design has been made on the basis of the elastic theory, using working loads and allowable stresses. It has been also based on the ultimate strength theory, using factors of safety on the loads, often termed as load factors. For most countries in Europe, recommendations and codes for prestressed concrete permit design on either the ultimate or the elastic basis. In this country, our engineers, being used to the elastic theory for reinforced concrete, tend to prefer the elastic rather than the ultimate approach. While the elastic theory works beautifully for prestressed concrete before cracking, the strength after cracking can only be determined by the ultimate theory. Unless care is exercised in setting up variable allowable stresses to suit different conditions, design by the elastic theory will yield divergent degrees of safety.

Ultimate design is based on a set of given load factors. Hence, structures designed by that method will possess the desired degree of safety, although they may be overstressed or understressed if judged by the elastic theory. However, ultimate strength design gives no assurance of proper behavior under service conditions, unless the design is checked by the elastic theory or a suitable factor of safety is chosen for each case.

(II) WHAT SHOULD WE DESIGN OUR STRUCTURES FOR?

Let us try to determine our basic requirements in design. First of all, it must be made clear that we should not design for fictitious or even real stresses unless we understand the significance of their values. Owing to the over-emphasis on stress calculations in our engineering training, our graduates do not know very much about real engineering although they do know how to compute stresses by theoretical and empirical formulas. They do not have sufficient idea of the actual behavior and strength of structures. The less they understand the structures, the more they tend to blindly follow certain codes of practice. It often ends up that we are simply designing our structures to satisfy certain code requirements which can be either too conservative or not sufficiently safe.

To experienced engineers, the real criteria for designing are:

1. We should design our structures so that they will perform satisfactorily under the usual service conditions. This means freedom from excessive camber, deflections, vibrations, cracks, etc.

2. We should design our structures so that they will possess sufficient strength to carry occasional overloads.

These criteria should be satisfied irrespective of stresses. While stress is sometimes a good indication of the fulfillment of these criteria, it is not always so, certainly not for prestressed concrete structures. In fact, to design by a fixed value of stress may result in undesirable, unsafe structures, even for conventional concrete. As an example, the writer recently witnessed several reinforced concrete slabs designed for 20,000 psi stress in the steel deflecting 3 inches on a span of 29 feet and costing the owner many thousands of dollars to have them repaired. The failure of reinforced concrete rigid frames of the Army warehouses is another example. (See CIVIL ENGINEERING, Feb., 1956, p. 45.) Although designed by the ACI code, they ruptured at the points of inflection even with no live load.

It will be interesting to note that many of our conventional structures designed in the usual manner are actually overstressed. Take a standard steel-angle connection for beams; the angles are always bent and overstressed. But such connections are used in all buildings. Again, compressive steel bars in reinforced concrete beams are often stressed to the yield point owing to the creep and shrinkage of concrete, so are some of the bars in reinforced concrete columns. But we do not worry about such overstress as long as the structures perform properly under service conditions and possess sufficient strength to carry overloads.

(III) PITFALLS WHEN DESIGNING BY ELASTIC THEORY

Good proportioning can be obtained by either the elastic or the ultimate theory if properly applied. When the elastic theory is used, we should not limit ourselves to a fixed set of allowable stresses. This is simply because designing for uniform stresses will yield structures of varying degrees of safety.

Consider a member concentrically prestressed and designed for no tension under working loads. If it is to serve as a direct tension member, it may have a factor of safety of barely unity. If it is to serve as a bending member, it often has a factor of safety as high as five. Is this criterion of designing for no tension a proper rule to abide by?

At the University of California, T beams and rectangular beams were tested to total rupture by excessive eccentric prestressing (Fig. 1). Both types were designed for no tension. The rectangular beams had a factor of safety of 4.0 while T beams of that particular proportion had a factor safety of only 1.3.

Again, a 5-inch slab 15-ft. square, was prestressed concentrically and tested (Fig. 2). According to the criterion of no tension, it can carry a live load of only 1 psf. But it actually carried a live load of 120 psf before cracking and 240 psf before rupture. Is no tension a proper criterion?

If designed for no tension, a prestressed concrete column will carry higher external moment when it is under higher compressive load (Fig. 3). However, the ultimate bending strength of the column is decreased by adding external compressive load. Should no tension be our basis of design?

If designed for no tension, bonded and unbonded beams will carry nearly the same load, but the ultimate strength of the unbonded ones is noticeably lower (Fig. 4). The addition of non-prestressed steel raises the ultimate strength of prestressed beams, but may not alter their elastic strength. Can we design all beams on the basis of no tension?

In England, a group of pretensioned beams were designed for no tension. But they cambered excessively as a result of elastic and plastic bending, so that they had to be rejected. Is no tension a sufficient criterion?

Similar examples can be cited for allowing a fixed compressive fiber stress. High

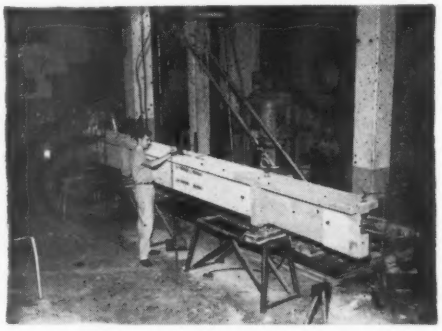
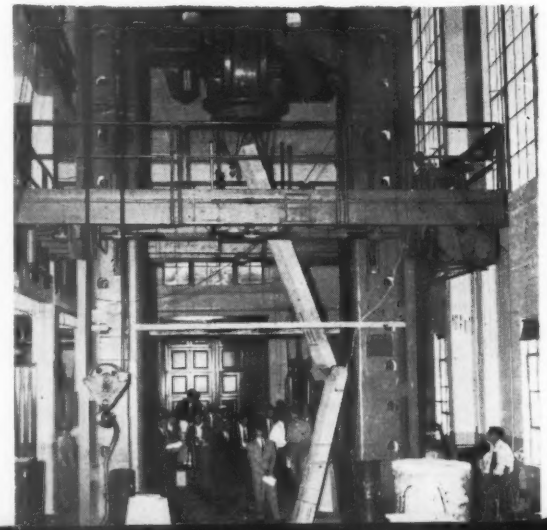


FIG. 1. T-beam tested to failure by excessive eccentric prestressing.

(University of California)

FIG. 3. Pre-tensioned column tested to failure under eccentric load.

(University of California)



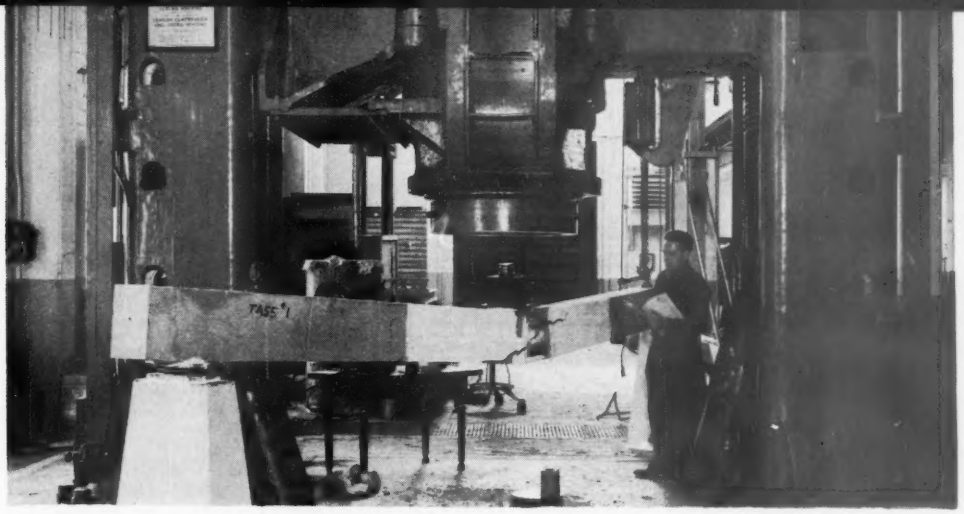


FIG. 4. Testing bonded and unbonded beams with and without non-prestressed steel (University of California).

compressive stresses at localized areas or in short beams may result in no objectionable deflections, while low compressive stresses over a major portion and in long slender beams may give real trouble, especially under the effect of creep. Hence to specify a fixed value of compressive stress may also err one way or the other.

Stresses are not the real controlling factors in design. The items we should look out for are: (1) camber under initial load and eventual creep, (2) deflection under actual service load, (3) possibility of cracking and whether it is objectionable, (4) vibration under moving load, (5) shortening of member under prestress, shrinkage, and creep, (6) ability to carry overload and the probability of overloads. While several of these items can be computed by the elastic theory, they cannot be controlled by a fixed set of allowable stresses. High stresses may not be objectionable while low stresses sometimes may. If this is so, what is the use of specifying stresses in design unless they can be comprehensive or flexible enough so as to cover all combinations of conditions? To specify a simple set of values would end up in structures that are either over- or under-designed.

(IV) Supplementary Checks For Ultimate Design

While ultimate design will definitely give the desired margin of safety, it may not yield a structure that will perform satisfactorily under service loads. A structure with adequate ability to carry overloads may camber, deflect, and vibrate under working loads. Hence, it is still necessary to check for its behavior under service conditions, if a structure is proportioned by the ultimate design procedure.

(V) Useful Strength

Rather than designing for the ultimate strength at rupture, another school of engineers considers maximum useful strength as a better criterion. The reason is that many structures may become entirely unserviceable long before they are loaded to the point of total rupture. A point of perceptible permanent set, for example, may be the most we would ever like to load a structure to. It is possible that, when such useful limit can be determined for structures, it could be a better criterion than the ultimate rupture strength.

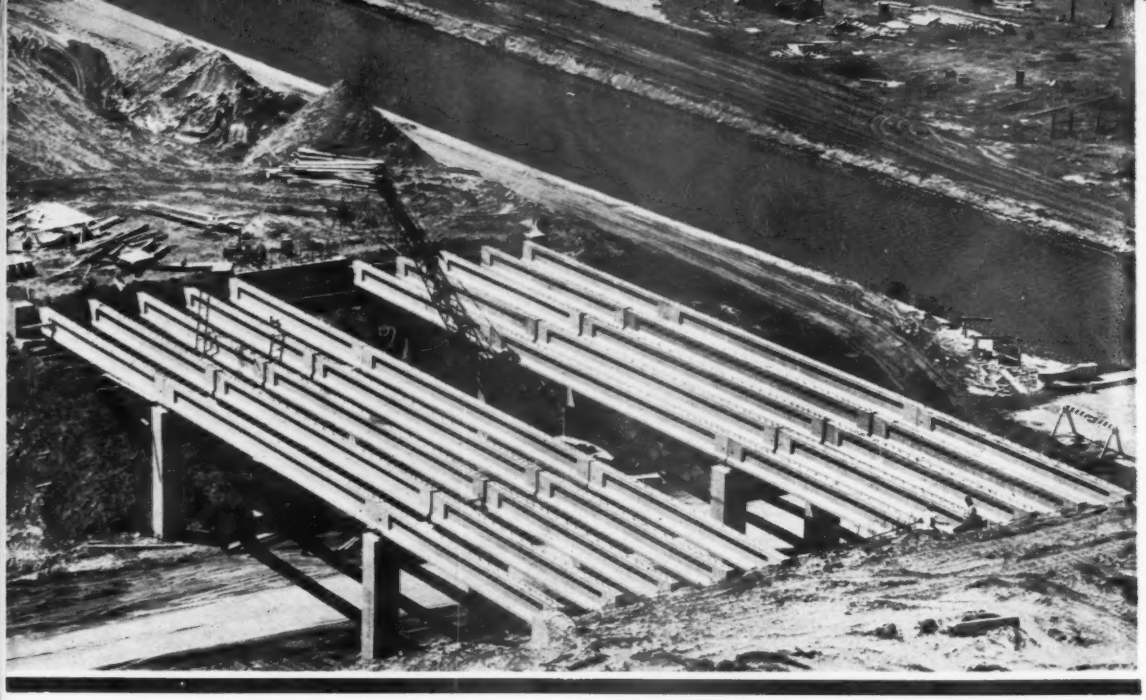
It is further noted that the so-called ultimate rupture strength is often the strength indicated by a single static loading directly to failure, the ultimate load may vary according to the speed of loading, the frequency of loading, the duration of loading, some of which may become a controlling factor under certain circumstances.

(VI) Conclusion

It is evident from the above discourse that elastic design on the basis of a fixed set of allowable stresses is entirely unsatisfactory for prestressed concrete. However, so many engineers are already familiar with the elastic theory, it may be difficult to abolish that approach. Hence, it is suggested that a flexible set of values be specified to guide the engineers, but not to restrict them. Structures so designed should be checked for factor of safety at ultimate strentgh, unless such factor is already known from similar designs.

Ultimate design with a proper set of load factors is a simple approach. Load factors or factors of safety must be determined for different loading conditions and for different types of structures. Where useful strength is known, design on its basis may be considered.

Whether designed by the elastic or the ultimate approach, the structure must be checked for possible excessive camber, deflection, cracking, creep, vibration and ability to carry repeated, continued, or impulsive loads whenever they are expected to occur. Actual analysis may not be required if it is known from experience that the structure will behave properly. Engineers should design prestressed concrete on the basis of its behavior and strength, not on the basis of arbitrary stress requirements.



Comparative Costs of Prestressed Concrete Construction and Conventional Types in Trestle Span of Florida Bridges

By W. E. DEAN, *Chief Bridge Engineer, State of Florida*

The low terrain of Florida with flat slopes and broad flood plains of inland rivers and the many wide tidal bays and sounds separating the mainland from the highly developed beaches are topographic features that require the provision of scores of exceptionally long bridges. Construction of new structures and replacement of old ones is a continuing procedure that has no ending. Many of these bridges have overall lengths more appropriately expressed in miles than in linear feet but most of them are essentially very simple structures. Water depths and foundation conditions throughout most of the State are such that adequate support for light spans can be provided by bents of friction or point bearing piles. This has resulted in a wide usage of rather short trestle spans. The Florida bridge of great length is usually a series of dozens, or perhaps hundreds, of identical short, simply supported trestle units.

Simplification and economical detailing of trestle construction is under constant study in the Florida Road Department. Any process or procedure likely to result in cost savings in simple bridge building is worth considering as a rather small saving on a single span if applied to a large number of spans can amount to an important sum. Prestressed concrete entered this field approximately four years ago and its use has repeatedly resulted in substantial economies in both the longer bridges with many identical spans and, lately, in the shorter structures of only a few.

Until recently, standard trestle spans up to about 45 feet in length, constructed near the coast with exposures to salt laden spray and atmosphere have usually been of reinforced concrete T beam type. For longer spans and for less critical exposures on inland bridges, composite I beam-concrete deck spans were standard. Spans with pre-

stressed members are currently being used as substitutes for both of these conventional types. All prestressed construction used to date has been composite and simply supported with the precast-prestressed beam carrying all dead load and the beam-slab combination resisting live load. This paper deals with this type only.

Many structural and maintenance advantages can justly be claimed for prestressed construction but it has been used in Florida work only on a basis of contract cost competition with one of the conventional types. To date, in every case, the prestressed alternate has won the competition on every job on which it was allowed. Our experience has been so consistent on this that the policy of designing alternate types is being abandoned for projects on which prestressed members are applicable. Lately we have had several important jobs with keen competition where no bid was offered on the conventional types.

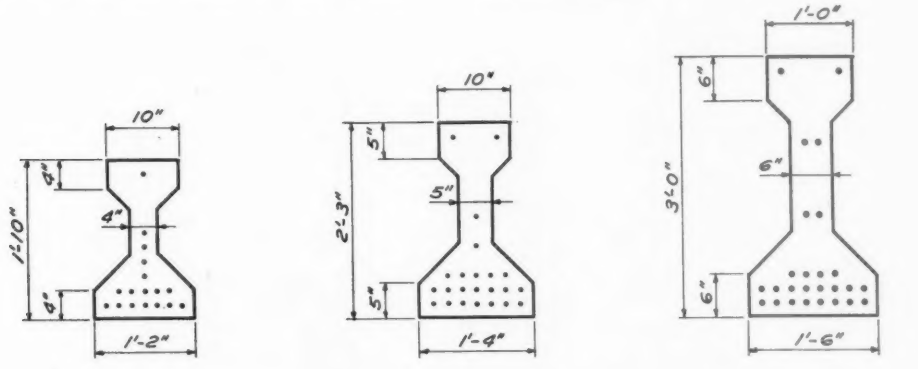
The use of prestressed members in Florida bridges commenced in 1951 when a design prepared by the Preload Company was allowed in competition with two conventional types for 3½ miles of trestle in the Lower Tampa Bay bridge. The prestressed design was bid at a saving of about 4%. Little further work in prestressed planning was done for a period of about two years during which time the Tampa job was tested and evaluated and found to justify both the structural computations and the Contractor's cost estimates.

In 1953 there was an acceleration in prestressed planning occasioned, first by the design of several long trestle bridges and secondly by the establishment of several large pretensioning plants to serve, principally, the building industry. This later development is of special significance as these yards have facilities for making many types and sizes of members economically in numbers of only a few or many. This has made the use of prestressed members practical and economical for the small jobs requiring only a few parts.

In order to keep the producer's capital investment in forms to a practical limit, the design of prestressed beams has been standardized insofar as possible. Figure 1 shows three sizes of beams, manufactured in the pretensioning yards, designed to accommodate all loadings and all span lengths from 25 to 60 feet. In each size, the cross section and end block details remain constant for various lengths and lateral spacing in the structure is varied to satisfy the requirements of span and loading.

Standard beams for post tensioning, as shown in Figure 2, have been developed for spans of 45 to 72 feet. These beams are an extension of the original design used for beams of the Lower Tampa Bay bridge. They are also used with the combined method of stressing where the straight tendons are pretensioned and the draped tendons are post tensioned.

Table 1 shows the results of bidding on ten typical projects of varied type, overall length and individual span length. A first cost differential, although of variable percentage, has been a consistent characteristic of bidding on these and all other jobs on which a prestressed design was offered. The superstructure cost includes all items above the substructure to, but not including, the handrail. The area upon which the cost per square foot is based includes the clear area of roadway and walks between the inside faces of railings.

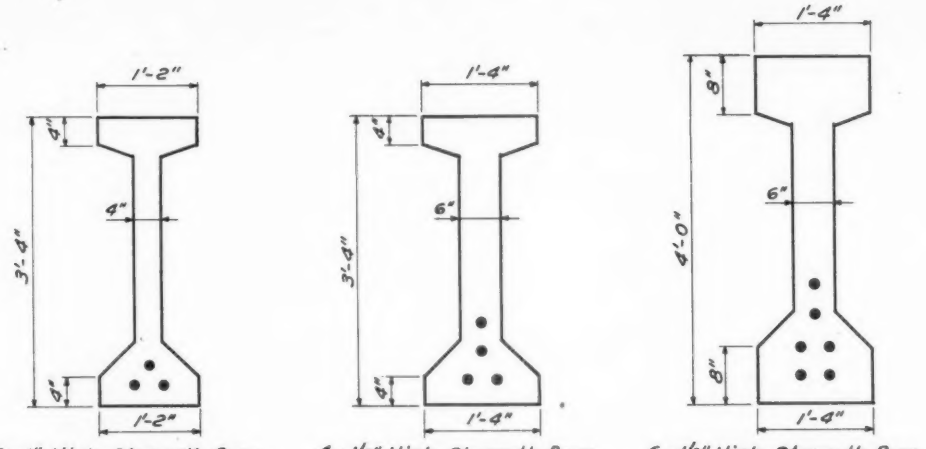


17~3/8" Strands @ 14,000#
TYPE A
Spans 25' to 30'

24~3/8" Strands @ 14,000#
TYPE B
Spans 30' to 45'

26~7/16" Strands @ 18,900#
TYPE C
Spans 45' to 60'

STANDARD PRETENSIONED BEAMS
FIGURE 1



POST TENSIONED BEAMS

Cross Sections at Mid Span

FIGURE 2

A 106-mile section of the Florida Turnpike is now under construction from Miami north. Engineers for the Turnpike Commission have adopted the Road Department standard pretensioned beams as alternates for I beams for all structures on which they are applicable. To December 15, 1955, bids have been received for 22 bridges on which the alternate was allowed. The bridges are principally grade separations or small structures of three or four spans with individual span lengths of 35 to 55 feet. On nine of the bridges no bid was received for the I beam alternate. On the remaining 13, the prestressed alternate was bid low in every case for a total saving of $93,000.

It might be appropriate to note some design features which tend to limit construction with prestressed beams. Curves, warps, skews or other geometric requirements which result in highly variable lengths and shapes may not be applicable to precast-prestressed construction on account of the high cost of forming. Members for such specialized construction can often be best furnished through the fabricating facilities of the steel industry. Just where the economic line can be drawn is questionable. In Florida practice prestressed beams have not been used in spans skewed more than 20 degrees.

Pretensioning, with the reinforcement in a constant position on a bed several hundred

TABLE 1

BRIDGE	NO. & LGTH. OF SPANS	RDWY WIDTH	WALKS	TYPE OF STRESSING	SUPERSTRUCTURE COSTS		ALTERNATE		DATE OF BIDS
					PER LIN. FT. OF BRIDGE	PER SQ. FT. OF BRIDGE	TYPE	COST/SQ. FT.	
Lower Tampa Bay	363@48'	28'	2@3'	Post.	$116.69'	$3.43	Reinf. T.	$3.53	1951
Cortez	50@48'	24'	2@5'	Combine	180.00'	3.18	Reinf. T.	3.38	1954
Escambia River	71@36'	28'	2@2'	Pre.	94.45'	2.95	Reinf. T.	3.26	1954
Hillsborough River	8@53'	28'	2@2'	Pre.	116.98'	3.66	I Beam	4.66	1954
Upper Tampa Bay	252@48'	26'	2@1'-6"	Post.	87.96'	3.03	Reinf. T.	3.59	1954
	20@72'	26'	2@1'-6"	Post.	166.67'	5.75	I Beam	5.75	1954
Myakka Valley (6 Bridges)	25@25'	24'	2@1'-6"	Pre.	90.41'	3.35	Reinf. T.	3.44	1955
Palm River	10@36'	56'	2@5'	Pre.	314.77'	4.77	Reinf. T.	4.85	1955
	1@45'	56'	2@5'	Pre.	351.11'	5.32	Reinf. T.	5.59	1955
Anclote River	7@45'	28'	2@3'	Pre.	144.44'	4.25	Reinf. T.	No Bid	1955
Manatee River	30@66'	56'	2@2'	Post.	272.75'	4.55	I Beam	7.32	1955
Palma Sola	72@48'	24'	2@5'	Post.	110.00'	3.24	Reinf. T.	No Bid	1955

feet long, is ideal for production line manufacture and minimizes the cost of stressing operation. Post tensioning with part of the tendons draped to offset the variable dead load moment is more efficient. Relative economies between the two methods will vary between different jobs and different Contractors. Limited experience on fairly large jobs where Contractors have had a choice between the two methods has indicated that for spans above about 45 feet, the extra labor in stressing the draped tendons is more economical than the extra material required when all tendons are straight.

The limiting economic span length for this simply supported composite construction is also difficult to define. From the table it will be noted that in 1954, a 72-foot span was bid at the same price with prestressed beams and with I beams. This span would then appear to be near the point of equal first cost for the prices prevailing at that time. Recent increases in the delivered price of fabricated steel have definitely increased this economic length. Note that the table shows 1955 bids indicating a cost differential of steel above prestress of about 50% in 66-foot spans. In fairness, it must be noted that these bids were by different Contractors and superstructure items other than the beams were involved. It would appear that possibly any span in which the length would not be economically impractical for simply supported I beams would be practical with precast-prestressed beams.

While this paper has been concerned only with construction using prestressed beams it might be stated that good use is being made in Florida bridge practice of prestressed bearing piles, sheet piles, handrails and other structural parts.

It can be definitely stated, in conclusion, that prestressing, which was little more than an interesting probability five years ago, has become a firmly established construction practice and one that will become increasingly important in bridge and building practice.

Prestressed Concrete Casting Beds *and Amortization*

By Douglas P. Cone, *President,*
Florida Prestressed Concrete Company, Tampa, Florida

As in any other business, depreciation schedules of prestressed concrete casting beds are an important factor in an economical operation. The question is often asked of the writer, how long do you set up your beds for write off? The answer, of course, is dependent on what type casting bed is involved and what is its prospects for future use.

It is generally agreed that the life of a casting bed is based on obsolescence rather than its physical endurance. Often the poor "prestresser" finds that a change in popular mode has left him with an expensive and obsolete bed on his hands that will not even serve as good rip rap in some out-of-the-way place. Frequently, a bed is set up for one particular job that has been designed by a well meaning engineer but for which it is impossible to find another home for the product after the job is completed. This type bed, needless to say, should be written off in one brief year. New methods, new forms and above all new ideas in the rapid growth of prestressing in the United States makes it extremely dangerous to set up a casting bed to be written off in a period exceeding three years. I have used three years as an average and predict that it will not be far from right when the last shot is fired.

Forms should not be neglected and it is understandable that this fast changing article is even more critical than the casting beds. This can be attested to by the piles of slightly used scrap iron in most producers' yards. If forms are set up for one particular job they should be written off over the period of that job. Rarely should forms be written off for over a period of two years unless they compose an integral part of a casting bed. Even then, the situation should dictate the policy.

Development of PRESTRESSED CONCRETE in bridge construction

By J. C. RUNDLETT

Bridge Engineer, Dept. of Public Works
Boston, Massachusetts

THE acceptance of a new design is generally not universal unless there is the urgent need of it for practical or economic reasons. Hence, development in one country will lag behind that in another and development within a country such as ours will not be uniform for the same reasons.

Thus, in prestressing, Europe and South America have made the greatest strides in this field spurred on by the scarcity and excessive cost of structural steel and the abundance and cheapness of labor. In this country the initial development was in the northeast during a critical material shortage, but the greater development is now in the south and west where the use of structural concrete is more universal. The natural tendency in these areas, because of relative economy and the climatic conditions, is to the extensive use of concrete and thence to the greater use of prestressed concrete.

This is not to say that concrete is not suitable to the widely changing climate of the northeast—it is—but there are other climates where exposed concrete can be expected to have a longer life. For instance, our New England climate is not suitable for the use of concrete piles in sea water, but Florida uses them successfully in all their long trestle work; therefore, the development and the use of prestressed concrete piles in that area is as natural as the greater development of prestressed bridges.

The use of prestressing in bridge work is by no means limited to new construction or, for that matter, to concrete. Inadequate structural steel floor beams can be cheaply strengthened by prestressing with cables or rods on each side of the lower portion of the beam, with the force applied against a plate welded to the end of the floor beam.

Inadequate concrete T-beams can be similarly strengthened to raise their capacity to the required loading (Figure 1). Such prestressing against an end plate and at a predetermined distance near the bottom of the beam can, if necessary, reduce the dead load stress in the reinforcing steel to zero and greatly reduce the compression in the top flange of the T-section, thus making available that stress range for live load.

In the prestressing of the concrete T-beam, tension may be developed in the upper ends of the beams, but you will recall that most older designs required about half of the bottom steel in a concrete beam be bent up and carried to the ends of the beam. This bent-up steel is in the convenient position then of being able to take the induced tension.

Development in this country has been, to a large extent, independent of European practice, one notable exception being the Walnut Lane Bridge in Philadelphia. The Magnel method stressing was used on this bridge where the wires are stretched two at a time and fastened with metal wedges.

Following the natural tendency of American engineers the casual comment was that there must be cheaper ways of post-tensioning stressing without having to stress the wires individually. In Europe, further developments had been made along this same line.

In England, Lee-McCall developed a cold worked high-tensioned rod with an ultimate strength of 145,000 p.s.i. These rods, which are available in sizes up to 1¼" in diameter, have tapered threaded ends and nut fittings which will develop the practical strength of

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(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on May 11, 1955.)

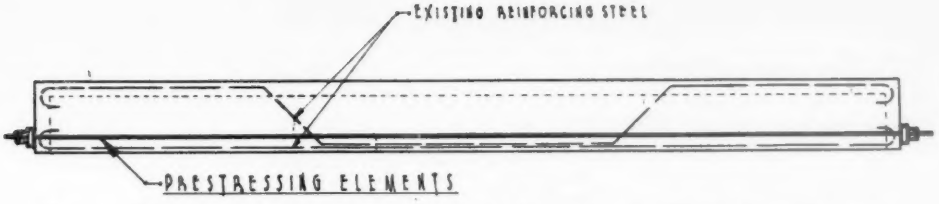


FIG. 1. Method of Increasing Capacity of Concrete Tee Beam by Prestressing.

the bar itself. The Stressteel Corp. of Wilkes Barre, Pennsylvania, has since taken over the manufacture of these rods in this country and are now in the process of developing a swedge fitting to replace the nut.

Freyssinet also developed a relatively inexpensive reinforced cone which allowed the stressing and anchoring of twelve 0.916" wires in one operation.

In America, Reobling developed a fitting for their bridge strands which made possible the stressing of the multitudinous wires in a 1-9/16" diameter cable at one time. These cables have an ultimate of 200,000 p.s.i.

The Prestressed Concrete Corp. developed a method of stressing six 0.25 inch wires by threading the wires through holes in a round washer with the head of the wires cold pressed into a button head which bore on the washer, thus allowing six wires to be stressed in one operation.

Another development along the same line was the cold working of two button heads, one at the end of the wire for the stressing operation, and the second near the end for anchoring.

A more recent development of rods has been made by the Tenchen Corp. of Boston. These rods, having an ultimate of 180,000 p.s.i., are heat treated with upset threaded ends which are rated at least 17.5 per cent stronger than the bar itself. The upsets are provided with standard threads and high tensile heat treated nuts. Because of limits imposed by the method of manufacture, the bars are available only in shorter lengths and have to be coupled for long spans.

Rather than continuing with long span structures, as might have been expected, the engineers in this country almost immediately concentrated on the design and production of smaller spans, probably with the thought uppermost in their minds that this is a production country, that to be sold it must be competitive, to be competitive it must be mass produced, and when mass produced it must be transportable.

The first United States Conference on Prestressed Concrete held at M.I.T. in 1951, with its open discussions of extensive European experience and practice and the more limited American developments, stirred the imagination of those in attendance to at least experiment with this new structural phenomenon.

The shortage of structural steel for bridges at about the same time gave the added incentive needed, and so the way was paved for the first new reinforced concrete design since the rigid frame came into prominence about 30 years ago.

Since 1951, with few exceptions, the development has continued in the moderate length transportable-sized beam class, trying out all various type of stressing elements, experimenting with pretensioning and post-tensioning, and with combinations of both, combinations of longitudinal and lateral prestressing, all seeking the simplest, cheapest and most practical method of stressing and the most favorable shape to obtain the lightest weight of beam.

DESIGN

One of the important features of prestressed design is the fact that the initial stresses induced at the time of prestressing are never again exceeded except under heavy over-loads.

Figure 2 made for the Hale Street Bridge indicates the stress stages in these beams under various conditions of construction and loading.

For the concrete, the initial compression stresses are 1877 p.s.i. in the bottom flange and 227 p.s.i. in the top flange. After allowance for shrinkage, creep, etc., the bottom flange is reduced to 1483 p.s.i., and the top flange stress is increased to 323 p.s.i. With the pouring of the slab the compression in the bottom flange is 910, and in the top

With the surfacing and design live load the compression in the bottom flange is reduced to 32 p.s.i., while in the top flange it is increased to 1398 p.s.i. With the application of two live loads there is tension in the bottom flange of 675 p.s.i. and the compression in the top flange is increased to 1684 p.s.i., which is still about 200 p.s.i. less than the initial compression stress in the bottom flange.

The initial stress in the steel at the time of stressing is 130,000 p.s.i. After loss of shrinkage, creep, etc., this stress is reduced to 110,000 p.s.i. With the poured slab and surfacing added, the stress in the steel is increased to 119,000 p.s.i. under the design live load, and to 123,000 p.s.i. under two live loads.

This is a simple demonstration of a manufactured beam which is prestressed well beyond twice its live load design during its construction. If there are then no visible defects or failures in the beam or in its prestressing elements there will be none under normal live loads or in loads far in excess of normal.

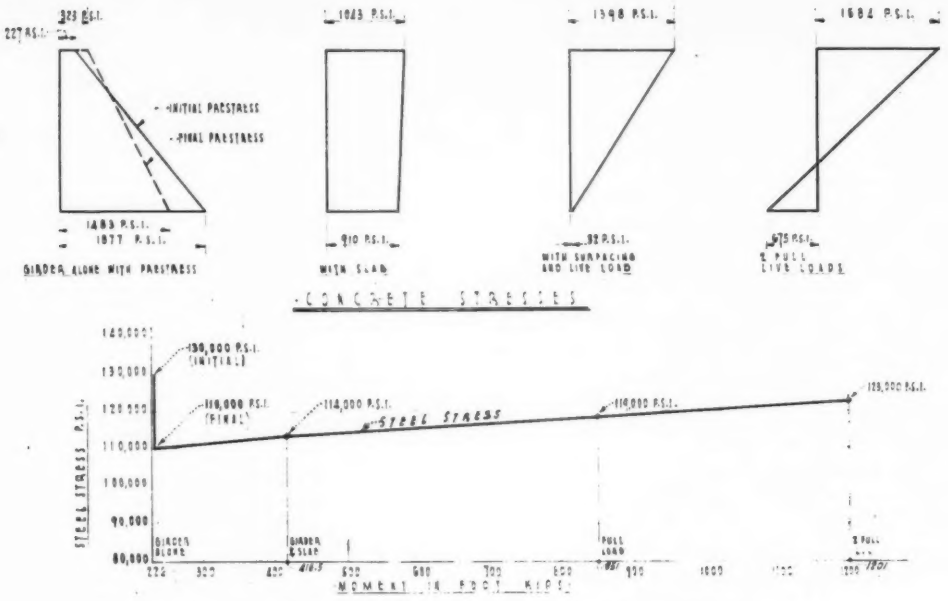


FIG. 2. Stress Distribution Curves—Hale Street Bridge.

The Department's designs have been very conservative, and I believe in the initial stages of such work that they should be. In the light of our experiences and tests and in view of the new specifications, our costs of this work can be materially decreased.

DIAPHRAGM DETAILS

Diaphragms are required on all prestressed work but, because of the necessity of having uniformity in precast members to make their production economical, the tendency is away from the requirements that they be placed at right angles to the beams except on a square bridge which, incidentally, is now an oddity.

In our early structural designs, diaphragms were believed to be of the utmost importance, that they must always be at right angles to the beams and that they should be made up of heavy beam sections with a depth of not less than 6" less than the depth of beam. It was felt that these members were the prime distributors of the loads from one beam to the other.

Subsequent tests have minimized the importance of diaphragms and have proved that the slab is an adequate distributor. Thus, the diaphragms become, in effect, more of a construction brace, and the skewing of diaphragms is then acceptable in prestressing.

On our Route 1 work the New England Concrete Pipe Company developed a cylindrical diaphragm stub at the diaphragm location which can be rotated to fit the skew of the bridge, thus obtaining the desired uniformity of details.

TESTING

As a part of our co-operative research program with the Massachusetts Institute of Technology many tests have been run, both at the Institute and at the New England Concrete Pipe Company, on beams selected from our different projects and on small-scale beams.

On the Endicott Street project at Danvers, three beams were tested, two with unbonded steel and one with bonded steel. You will notice from the load deflection diagram of these tests that up to the point of cracking, these three beams behaved in an almost identical manner but that beyond this point the bonded beam showed less deflection. This behavior convinced us that post-tensioned work should always be grouted in bridges because of the ability of that type of beam to withstand better the effects of overloads, and further, because of the added protection of the steel against corrosion.

On the Newbury-Newburyport project, one beam was selected from each of the four bridges constructed. Loading to simulate live load and the dead load of the surfacing was applied at the third points by means of two hydraulic jacks (Figure 3). Loads were applied in stages in each test to a point beyond the cracking load, the loads were then released to zero and then re-applied till failure.

Cracks as they appeared on the beams are indicated on the curves of Figure 4 by numbers under the load at which they occurred. You will note in all cases that they are far above the qualifying conditions of

two times the live load. The appearance of these cracks was noted by an observer and checked with a sonoscope by placing the poles diagonally on opposite sides of the bottom flange near the point where the crack was expected to appear.

Strain gauges were not used in these latter tests except on the Lee-McCall bars. A controversy had developed because of the brittleness of this type of steel, of the effect of notch action in the threads, and of the possibility of greatly increased stress in the bars by eccentric loadings due to inaccuracies in the setting of the bearing plates. Also, while it was known under normal loads that the stress in the bars and fittings would never exceed the initial stresses, the effectiveness of bond by the grouting of a smooth rod was not known.

Four SR4 strain gauges were attached to the four bottom bars near the end plate and as near the nut as possible to measure the changes in stress at this point as the loads were applied. These gauges indicated that the grouting did give sufficient bond to the extent that there was no change in stress at the ends until the loads had reached the equivalent of four live loads. At this point the bond in one bar became ineffective, the stress increased rapidly, and the member failed through the threaded portion just inside the nut. Unfortunately, the test was then stopped. We regret now that it was not run to destruction.

Tests run at M.I.T. on 0.6" diameter strands further indicated that bond achieved either by grouting or by tensioning the strands prior to casting and supplying end anchorage for the strands after release increased the strand efficiency and ultimate moment by 20 per cent.

A further test was run on a composite steel beam and a slab, with the same span and design conditions as the Freyssinet prestressed beam to compare the action of these two designs. Both load deflection curves have been superimposed on Figure 5 showing an intersecting comparison, and indicating that the prestressed design was superior in withstanding extreme overloads.

The most impressive results of these tests were the flexibility of a prestressed concrete beam, commonly thought to be a rather rigid member. The Scotland Road Bridge, with a span of 65 feet and designed for an H20 S16 loading, had a total deflection of 28" at failure. The Freyssinet beam, with a span of 64 feet and designed for H20 loading, had a total deflection of 24", while the LeeMcCall beam, having a span of 58', and the Prestressed Concrete Company's beam, having a span of 59', and both designed for H20 loading, had a deflection at failure of about 16".

The next most impressive result was the almost complete recovery of the beam after the application of about four live loads and the complete closure beyond recognition of any cracks.

These tests, I believe, do not particularly indicate any advantage of one system of prestressing over another, but they do prove beyond any question of doubt that prestressing is an excellent bridge material, that it is structurally sound, and that no one need have any qualms about using it.

FIG. 3. Testing Prestressed Beam.

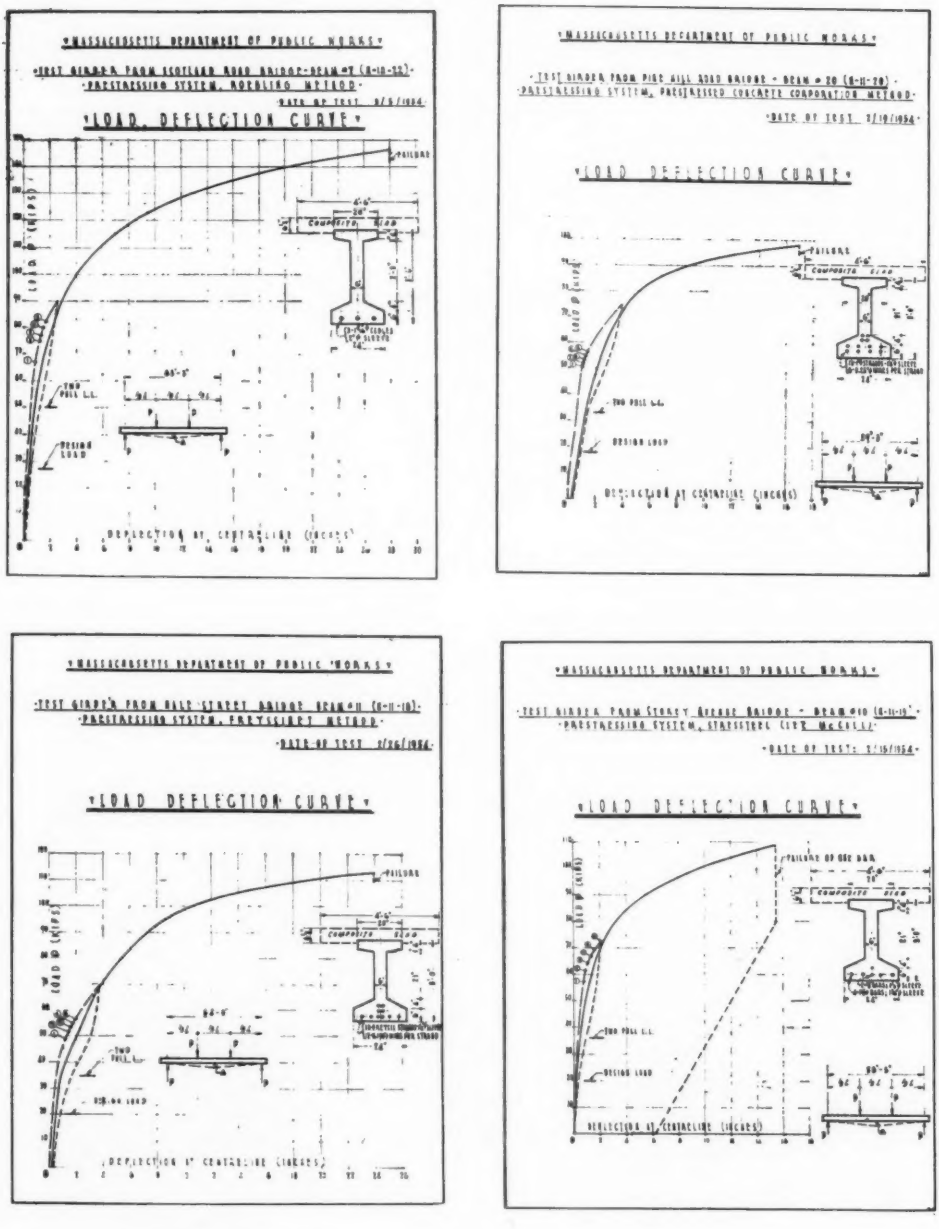


FIG. 4. Load Deflection Curves.

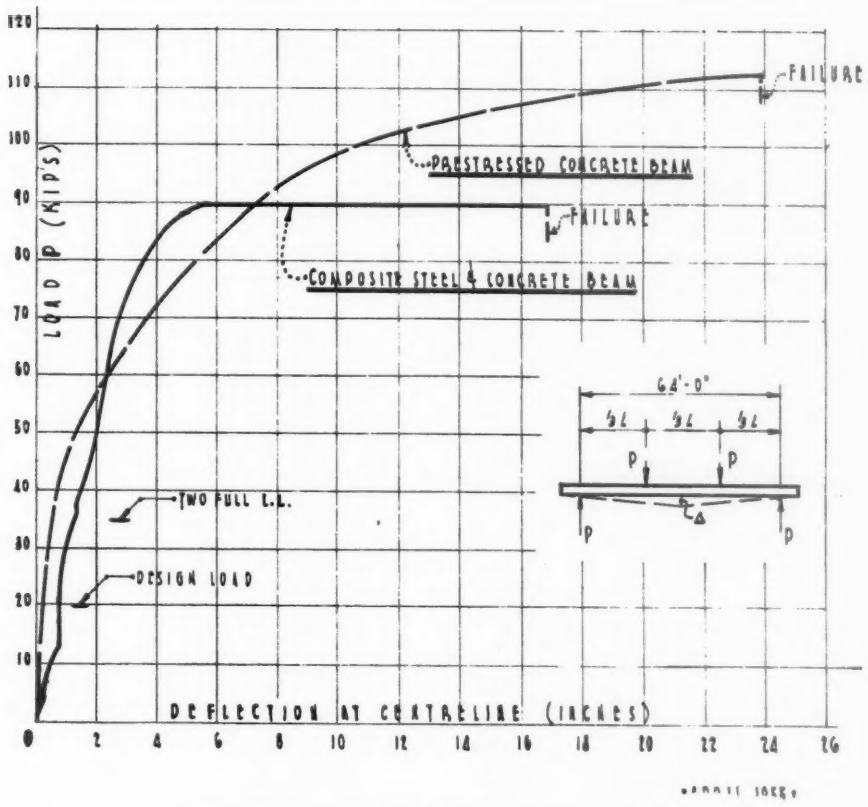


FIG. 5. Comparison of Load Tests—Prestressed Concrete Beam—Composite Steel and Concrete Beam.

EXAMPLES OF PRESTRESSED BRIDGES

Turkey Creek Bridge—Tennessee

I believe that the first prestressed bridge in the United States that was open to travel was the Turkey Creek Bridge in Tennessee, designed by Bryan & Dozier and built in 1950. This bridge was of precast block construction similar to the wooden block models used to illustrate prestressing. In this type of design, generally three types of blocks are required; the end or bearing blocks; the intermediate blocks, which retain the cables in position near the bottom of the beam until they are bent up; and the standard plain blocks.

John R Bridge—Michigan

Another similar design to the Turkey Creek Bridge is the John R. Bridge designed by Johnson & Anderson and built over the Red River Drain in Michigan in 1951. On this bridge the blocks were constructed with lips and the space between blocks filled with grout. All bridges of this type are stressed with cables after the grout has sufficiently set. Spans have been constructed up to about 32′ and a continuous bridge has been built by using reinforcing steel in the poured slab over the supports.

This type of construction has been very economical and has many advantages, but has not been used extensively. There is still a reluctance to accept beams that are not poured monolithically and, for my part, I feel that they do not have a place in our climate.

Block construction is the reason for the omission of a cracking load requirement in the most recent specification because, in this type, computations for cracking are largely meaningless.

Wyoming County—New York

A later development of multiple units in a beam has been made by Mr. Felix Ramsey, Superintendent of Highways in Wyoming County, New York, in an effort to produce beams using his own limited forces and equipment.

For ease of handling, each beam is precast in three sections with the size of the unit limited by the capacity of his cranes. Each individual section is reinforced sufficiently to allow handling and transporting.

The longest span which he has developed to date is 52 feet, and was cast in about 17′ lengths with two end units and one center unit. The section is T in shape, 3′ deep and has a flange width of 30″.

Each section is poured against the other in the shop to insure an accurate fit in the field. The precasting is done indoors in the winter season and stored there until time for erection. The units are then taken to the site, placed in position on the abutments and on two temporary bents, the wires are threaded through pipe sleeves, stressed, anchored and grouted. The Freyssinet system is being used.

Arroyo Seco Bridge—California

The Arroyo Seco Pedestrian Bridge was built in California in 1950. This structure has a span of 110′, a width of 8′ and is the west's first prestressed bridge. The beams are 5′-8″ deep, with 10″ webs, were cast on the adjacent roadway and moved into place with three cranes. The stressing was done by the Prestressed button head method.

Walnut Lane Bridge—Philadelphia

The first bridge placed under construction in the United States was the Walnut Lane in Philadelphia, which was completed in 1950. It contains three simple girder spans, a center span of 155′, and two end spans of 74′ each. The girders are I-shape, 79″ deep with 52″ flanges. In the center span the flanges are butted, but in the end span the beams are placed 8′-8″ on centers and the slab poured in place. As noted before this bridge was tensioned by the Magnel method, two wires at a time. The beams were cast on falsework at the bridge site and were moved horizontally into position on the foundation.

Endicott Street Bridge—Danvers

In the fall of 1951, Massachusetts started construction on the first prestressed bridge to be built in northeastern United States. This was the Endicott Street Bridge in Danvers over the Boston and Maine Railroad. This bridge has a span of 27′-4½″. The beams are of an inverted T-shape with the bottom flanges butted and the area between and over the webs filled with concrete, making in effect a composite slab. The beams are 14″ deep with a 12″ flange and were stressed with three 0.6″ galvanized wire strands, two straight and one parabolic, and all unbonded. The 47 beams were cast in the Providence shop of the New England Concrete Pipe Company, about 65 miles from the bridge site. They were transported by truck over the road and erected all in one day.

The particular advantage of this type of bridge is that no falsework or forms are required for the pouring of the deck slab except as may be necessary for the copings and curbs.

While this type of beam has not been extensively used on bridge work it was used with pretensioned strands on Pier 57 in New York very successfully.

Pretensioned Beams—Pennsylvania

In 1951 the Concrete Products Company of Pottstown, Pennsylvania, having previously manufactured precast inverted channel-shaped units, conventionally reinforced, became interested in prestressed concrete. Their natural interest was in production and they became convinced at the start that only by pretensioning could prestressed work be mass-produced and made competitive.

Pretensioning requires the setting up of a long bed with end blocks, against which the multiple wires or strands which extend the full length of the bed can be stressed. The concrete is then poured around the stressed elements into as many beams as can be accommodated on the bed.

With the time element still in mind, the concrete is cured by the vacuum process and when it has reached a predetermined strength the jacks are released and the wires cut, thereby transferring the induced stress in the strands to the concrete. The net result is a prestressed beam produced with the minimum of fittings, fixtures and man hours.

The beams which they now produce, Figure 6, are standardized with a 3′ width and are butted when in place on the foundation. In order to save concrete, and consequently weight, the beams are hollow.

For spans 18′ to 36′ in length, beams 17″ to 21″ in depth are used, and for spans from 38′ to 50′, 33″ depth beams are used.

In the initial development of pretensioning ¼″ strands were called for, but the tendency now is to increase the size, thereby reducing the number of units required. The size has been increased to 3/8″ with an occasional 7/16″, and the limit, of course, will be that size which can adequately and safely develop the bond of these high stress units.

Beams 36″ deep are now being produced, and plans are being made for beams 42″ deep and 70′ long.

Florida—Tampa Bay

No paper on prestressing would be complete without a reference to the original Tampa Bay Bridge in Florida which was the first and best example of mass-produced prestressed concrete units for bridge work in this country.

Florida is the ideal location for concrete pile trestles and their lengths are measured in miles rather than in feet. The prestressed work on this particular bridge is part of the 15-mile long causeway linking St. Petersburg and the West Coast of Florida. The structure has a roadway width of 28′ and two 3′ sidewalks.

Over three miles of this bridge is of prestressed construction and it is divided into four sections, one of 5800′, one of 10,900′ and two of 325′ each, with spans of 48′.

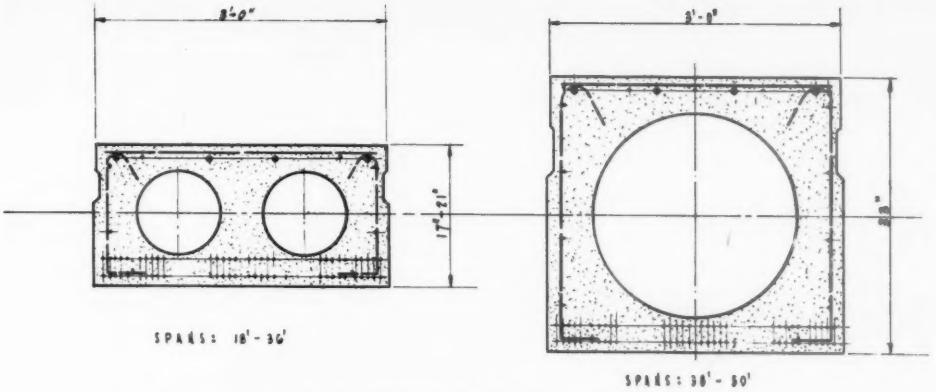


FIG. 6. Typical Pretensioned Beams.

2,178 beams were required.

Six casting beds, each large enough for the manufacture of 12 beams, were constructed on a spit of made ground projecting into Tampa Bay. These beds were arranged for a continuous operation of casting and stressing. The beams were cast, the forms removed in one or two days and transferred to another bed, and the beams stressed on the seventh day if the cylinder breaks indicated 4,000-pound concrete. The beams were stressed with LeeMcCall bars, then loaded onto barges, towed to the site and transferred to the bents with a floating crane.

Newbury-Newburyport Bridges — Massachusetts

Massachusetts has to date constructed 7 Prestressed Bridges including the one on Endicott Street. The other six made use of an I section with this type of beam substituted for a steel I beam of about the same depth and spacing and calculated for composite action under live load. All of the beams were manufactured by the New England Concrete Pipe Company.

Of particular interest are the 4 bridges built on the Newburyport Turnpike in Newbury and Newburyport, each making use of a different type of stressing element.

All the beams are of I shape, Figure 7, having a bottom flange width of 24", a top flange width of 20" and a 6" web. Three have a depth of 3 feet and were designed for H20 Loading while the fourth, the Scotland Road Bridge, had a depth of 3'-8" and was designed for an H20 S16 loading. The general spacing of beams is about 4'-6".

The Scotland Road Bridge carrying Route 1 has a span of 65' and contains 24 beams stressed with three 1-9/16" strands furnished by the American Steel & Wire Company. The Hale Street Bridge, having 2 spans at 64' contains 18 beams stressed by the Freyssinet system each with 10 prestressing elements of twelve 0.196" diameter wires. The Storey Avenue Bridge having 2 spans at 58' contains 36 beams stressed by the Lee-McCall system using four 1⅛" straight bars and two ⅞" draped bars. The Pine Hill Road Bridge having 2 spans at 59' contains 20 beams stressed by the Prestressed Concrete method using twelve 6-wire units of .25" wires.

All of the beams were grouted. The original specifications called for a different type of enclosure tube for the wires and strands but the contractor used a flexible

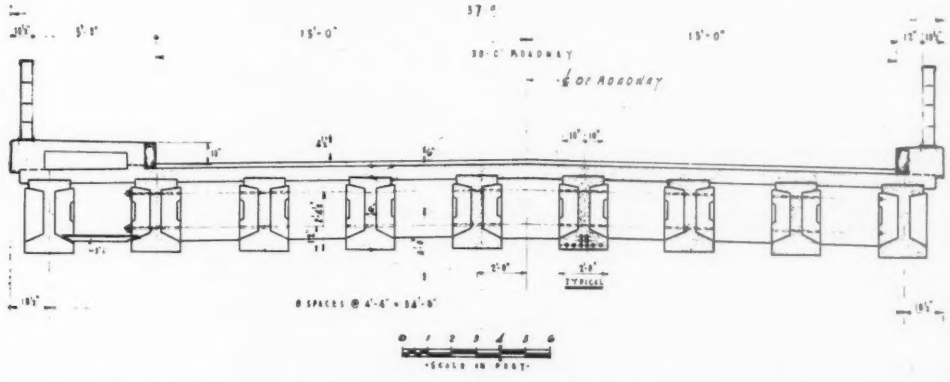


FIG. 7. Cross Section—Hale Street Bridge.

metal hose for all beams. This allowed the coiling of all the units in the hoses before shipment except for the Lee-McCall bars.

For the 1-9/16" strand, a 2" diameter hose was used; for the 12 wires of the Hale Street Bridge, a 1¼"; for the 1⅛" diameter and the ⅞" diameter bars of the Storey Avenue Bridge, a 1¾" and 1½" respectively; and for the 6 wires of the Pine Hill Road Bridge, a 1¼" hose was used.

The sections cut from the various test beams bear evidence to the success of the grouting operation.

These beams were fabricated out-of-doors during the colder winter months in a prepared bed and shelter. The bed, with width enough for 2 beams to be constructed at one time, was enclosed with two 4" diameter concrete pipe side walls over which a sectional wooden roof was placed and the ends were enclosed in canvas after the pouring and during the curing.

All the concrete was air-entrained high early with a strength of 5000 p.s.i. and the beams were all steam cured.

One set of steel forms was used in the early stages but after the letting of the Bridgewater contract for 24 more beams, a second set of forms was made.

The diaphragm stubs on these beams as previously described were of a turret type and were spaced equally on all beams, and the end bearing blocks were identical so that the variation in the forms was only the length between the last diaphragm and the end block on all beams and for the depth on one.

The complete manufacture and stressing of a beam was on a 2-day schedule with some interruptions.

A cage containing all the reinforcing steel, stirrups and stressing elements was set up on one side of the bed so that no time was lost in the bed during the fabrication of this unit.

During the pouring of the concrete both internal and external vibrators were used with the net result of perfect beams with no honeycombing, except on one beam made from a new shipment of cement which beam was subsequently destroyed and replaced.

To say that the work progressed without incident would be far from the truth.

The Lee-McCall bars which were shipped in bundles from England arrived unbundled and bent from careless handling. Most of these could be used but a few were replaced. Carelessness in stressing one of the first bars caused the bar to break and the jack to fly, breaking the operator's leg.

Many of the Freyssinet cones arrived in less than perfect shape and had to be replaced and some slipping developed after the stressing, but the amounts were insignificant.

The cold formed button heads on the wires caused some concern because of longitudinal splits. The manufacturer assured us that this was not a weakness and failure of the wire and not the head on subsequent tension tests proved this point.

While the cables for the Roebling Type were prestressed, the coiling of the cables in the tubes had disturbed the lay so that in the first stressing, the strain was not proportional to the stress. This difficulty was overcome by partial stressing and relieving before final stressing.

On our coldest days, when the bed was uncovered, a crack developed in the middle of the top flange apparently due to the rapid change in temperature. This crack in no case extended through the bottom flange and when the beam was stressed the crack closed completely even though camber was induced due to the stressing.

The beams were hauled over the road with one end on a truck and the other end on a dolly and in order to give lateral stiffness, two were shipped at a time. The carrier violated his hauling permit in length and in load and some time was lost in arranging for a new permit.

To cap the climax some one, probably a structural steel man, set fire to a load of hay stored under the Storey Avenue Bridge at 4:45 one morning (Figure 8). Some concrete was spalled on the beams, but load tests indicated that there was no structural damage and that the beams were satisfactory.

It is in this manner that experience is gained and it is for these reasons that some people feel justified in letting others do the experimenting.

Briefly, I would like to describe a few of the odd shapes and methods that have been used in prestressing.

Hubbard Street Bridge—Connecticut

Connecticut's first prestressed bridge was on Hubbard Street under the Hartford-Glastonbury Expressway.

It has a span of 52'-6" and is 90 feet wide. There are 20 T-shaped beams in the section, each 2'-6-½" deep with a flange width of 30" and a stem width of 15". The beams are spaced 4'-6" on centers and are topped with a poured concrete slab which was prestressed transversely.

The superstructure contract was let separately and no specific system was designated, but the steel area, positioning and total tensioning force was given. The Freyssinet System was used.

Atlantic Street Bridge — District of Columbia

The District of Columbia's first prestressed bridge was on Atlantic Street over the Oxon River. This bridge has a span of 74'-9" with provision on the deck for a 40' roadway and two 6' sidewalks.

The nine beams in the structure are 3'-0" x 3'-3-½", have a hollow core, and are spaced 6' on centers and are topped by a conventionally reinforced slab.

The beams were cast near the site and were stressed by the Freyssinet System.

West Mississippi Avenue Bridge — Denver, Colorado

Another interesting development was the bridge built in Denver on W. Mississippi Avenue over the Platte River. This structure

FIG. 8. Storey Avenue Bridge.

is a concrete pile trestle having 4 spans at 50 feet with provisions on the deck for a 40′ roadway and two sidewalks.

The beams have tapered sides with a re-entrant seat for the slab and are spaced 9′-3″ on centers. They are 3′ deep, 3′-6″ wide, contain a 24″ economy hole and were stressed by the Freyssinet System.

The slabs are precast and pretensioned, and finally the structure was laterally post-tensioned.

Two similar examples of fairly long span prestressed construction are the Willow Creek Bridge on the Columbia River Highway in Oregon, Shawan Road Bridge on the Baltimore-Harrisburg Expressway in Maryland.

Willow Creek Bridge—Oregon

The Willow Creek Bridge (Figure 9) has 3 spans at 98′ with provision on the deck for a 30′ roadway. There are 5 modified I-shaped beams in the cross section, spaced 7′-2-¾″ on centers. These beams are 5′-4″ deep, have a 7″ thick web, a bottom flange 2′-2″ wide and a top flange 4′-0″ wide. The slab between the top flanges and diaphragm stubs is cast in place and the entire deck laterally prestressed. The Freyssinet System was used.

Shawan Road Bridge—Maryland

The Maryland Bridge has a span of 100 feet and a roadway width of 30′ and two safety walks. It spans the expressway and was chosen as an experiment in contrast with two other 3-span steel stringer bridges on the same project.

There are 9 beams in the cross section spaced 4′-1-¾″ on centers. The beams are 5′ deep, have an 8″ web, a bottom flange 20″

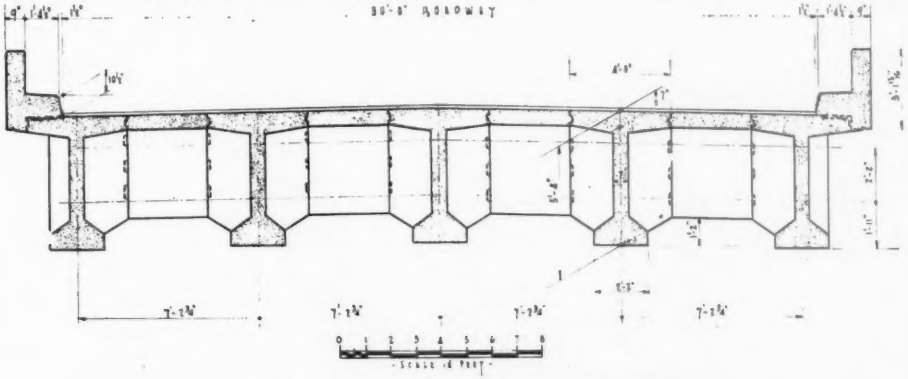


FIG. 9. Cross Section—Willow Creek Bridge.

wide and a top flange 3′-8″ wide. The poured slab between the top flange and the diaphragm stubs in this case was only 5¼″ in width as compared to a width of 3′-3″ on the Willow Creek job.

The Freyssinet System was used on this structure and the bridge was laterally prestressed.

Both of these structures made use of the top flange of the prestressed beams as a part of the structural roadway supporting slab with a bituminous surface for pavement. This type of construction cuts down the amount of concrete work in the field and results in an economical deck. In Massachusetts the deck would have a 3-ply membrane waterproofing course under the surface for protection of the slab.

My own feeling on the matter is that in New England the prestressed units should have the added protection of a poured top slab.

Future Farmers of America Bridge—West Virginia

Another example of pretensioning is the Future Farmers of America Bridge in West Virginia. The designer of this structure departed from the I and the square or rectangular section and chose an inverted U section with the prestressing element located in the vertical legs. This bridge has a span of 65 feet, is designed for an H20 S16 loading and has a depth of 2′-8″.

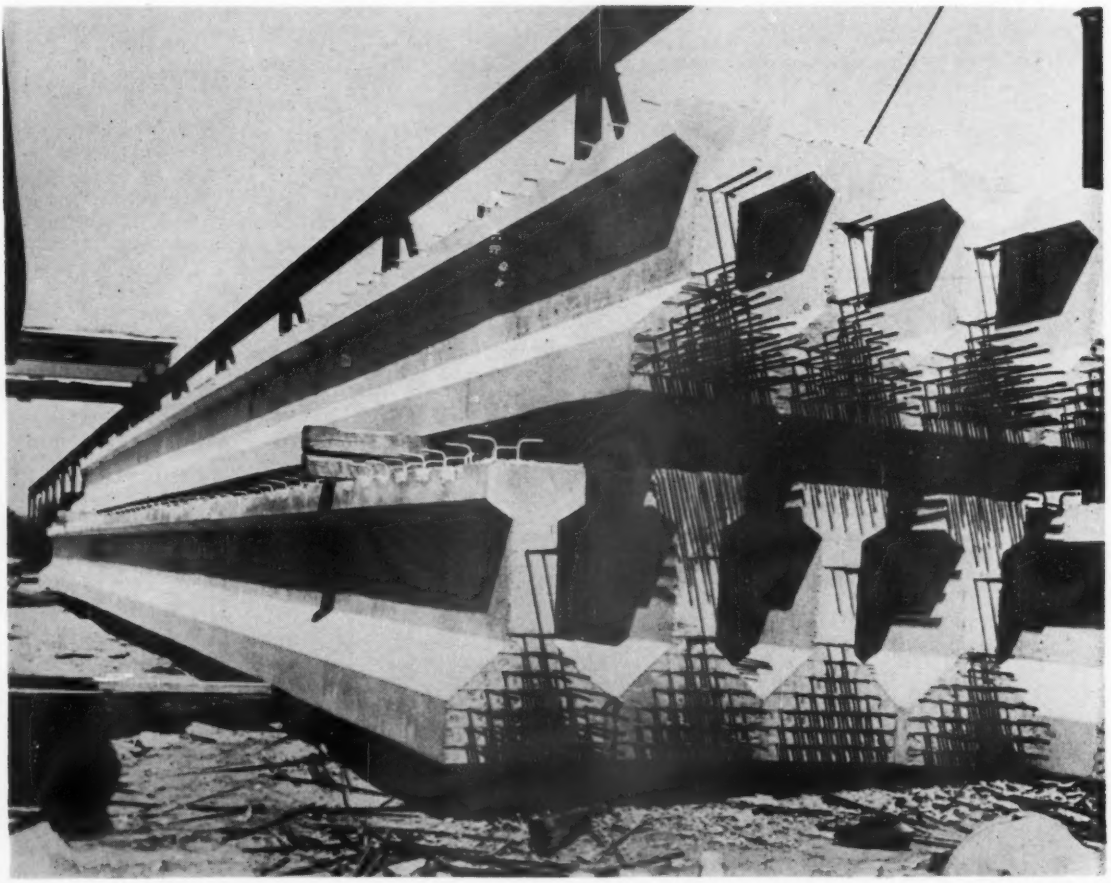
The precast sections are all 3′ wide and of 2 basic sections; the main roadway sections and the curb sections. These units are butted and bolted together in place and the keys near the tops of the units are grouted. Only the installation of a wearing surface is necessary before the bridge can be opened to traffic.

This is an excellent design but in climate such as ours an application of 3-ply membrane waterproofing would be required under the surfacing. Without it we would be faced with the seepage of water through the butted section and the deterioration of the concrete due to the many cycles of freezing and thawing.

Garden State Parkway Bridge—New Jersey

Twelve bridges on a recently completed section of the Garden State Parkway in New Jersey were of precast and prestressed concrete. The final plans called for the 28 exterior beams to be rectangular and post-tensioned and the 190 interiors to be of a modified I shape and pretensioned, Figure 10.

FIG. 10. Garden State Parkway Beams.



The spans varied from 39 to 60.4 feet but the beams were all made of two basic cross sections with the prestressing elements varying according to the span. The interior beams are 33″ deep with a 6″ web, 12″ wide top flange and a 19″ bottom flange. Twenty to forty ⅜″ strands were used in these members. The exterior beams are 33″ deep and 16″ wide and are reinforced with units of ten ⅛″ wires of the button head type.

These beams were manufactured in three plants about 30 miles from the bridge sites. High early strength cement was used, the concrete was steam-cured, and the beams cast in a two-day cycle, identical with the procedure used for the manufacture of the Massachusetts beams.

The Formigli Corporation, manufacturers of these beams, are now prepared to continue their production for other structures up to 60′ spans within hauling distance of their plants.

Lake Pontchartrain Bridge—Louisiana

The designer's dream of precasting and prestressing an entire span of a bridge is trestle type structure having 2237 spans of 56′ each with provision on the deck for a about to come true in Louisiana with the construction of the Lake Pontchartrain Bridge on the Greater New Orleans Expressway. This bridge, which has an over-all length of approximately 23-¾ miles, is a 28′ roadway.

A typical span, Figure 11, is composed of 7 modified I-shaped beams 4′ deep with a 6-¾″ top slab. The 7 beams and the slab are to be precast monolithically as a pretensioned unit. The 5 interior beams are to be stressed with thirty-five 5/16″ diameter wire strands, and the exterior with thirty-three 5/16″ diameter strands. The total weight of one span is about 180 tons and it is to be supported by 54″ prestressed concrete piles.

Hampton Roads—Virginia

On October 8, 1954, bids were received for the construction of a concrete pile trestle type prestressed structure for the Hampton Roads Bridge in Virginia. Alternate bids were requested for 261 post-tensioned beams versus steel stringers for the 80′ spans, and for post-tensioned versus pretensioned for 896 beams in the 50′ spans.

For the 80′ spans, six 1-⅜″ diameter bars were indicated for the 4′-7″ deep beams. For the 50′ spans, four 1-⅜″ diameter bars were shown for post-tensioned and thirty-six ⅜″ strands for the pretensioned alternate. Both of these latter types were 3′-7-½″ deep but a wider beam was required for the pretensioned members to provide ample space for the greater number of prestressing units. 24″ prestressed concrete piles were specified for the bents. The low bid submitted was on the prestressed beams and the final selection was for 50′ pretensioned spans for the entire project.

Tampa Bay Bridge—Florida

On September 30, 1954, alternate bids were received for the construction of a bridge adjacent to the Gandy Bridge over Tampa Bay in Florida. This proposed structure contains 252-48′ spans and prestressed construction was bid as an alternate to conventional reinforced concrete T Beams. There are also twenty 72′ spans with prestressed construction bid as an alternate with steel I beams.

For the 1505 concrete piles, prestressed or a standard reinforced type are the contractor's choice. An alternate type of prestressing was also indicated. For the 72′ spans, six 1-⅜″ bars were indicated for a post-tensioned beam while the alternate was for twenty-four 7/16″ diameter straight cables to be pretensioned and two 1-⅜″ diameter draped rods to be post-tensioned. For the 48′ spans, four 1-⅜″ bars were shown for the post-tensioned beam and twelve 7/16″ strands for the pretensioned in combination with two 1-⅜″ diameter draped rods to be post-tensioned.

The low bidder submitted a price on only the prestressed design and for that design post-tensioning was selected on all beams using Stressteel bars.

Subsequent to this, another trestle 3000 feet long was bid and the type chosen, which was again the contractor's option, was for combined pre- and post-tensioning, using strands for the pretensioned straight members and Freyssinet units for the post-tensioned draped members.

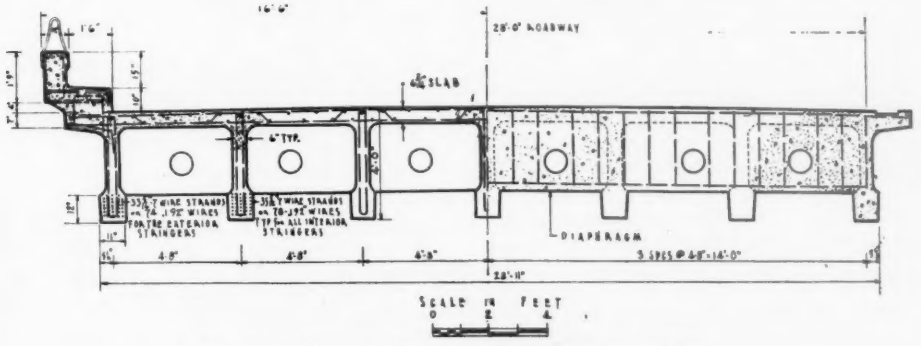


FIG. 11. Cross Section—Lake Pontchartrain Bridge.

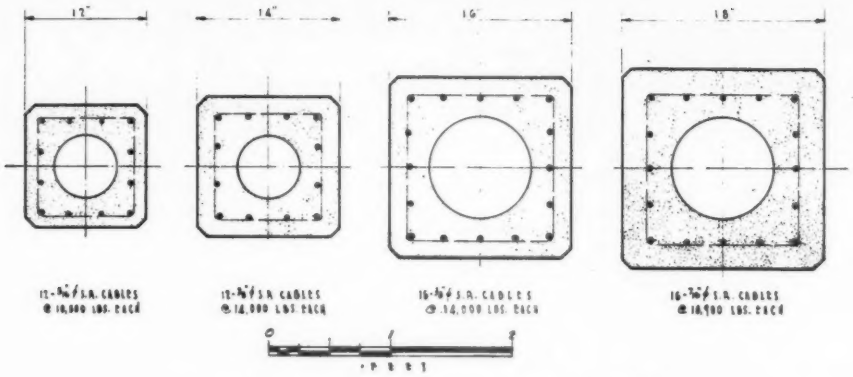


FIG. 12. Typical Prestressed Concrete Pile Details.

Prestressed Piles

The prestressing of concrete piles using the pretensioned method is rapidly gaining in favor. One of the disadvantages of long conventionally reinforced piles is the extreme care that must be exercised in handling and in driving because of cracking. Prestressing is the panacea for this. With the concrete in compression, cracking is eliminated in handling and any shrinkage cracks which develop after the pouring are permanently closed.

All the concrete pile projects in Florida bid to date have called for prestressed piles as an alternate to conventionally reinforced piles with the choice being the contractor's, and invariably prestressed piles have been used.

The pile sections in that state have been standardized for 12", 14", 16" and 18" square sections, reinforced with twelve ⅝" strands, twelve ½" strands, sixteen ½" strands, and sixteen 7/16" strands, respectively, with maximum lengths for these sections of 75', 85', 95', and 107' (Figure 12.)

Continuous Structures

Continuity is lagging in this country. There is a reluctance to duplicate or even approach the work of Dr. Fritz Leonhardt of Germany whose continuous design allows the stressing of an entire bridge in one jacking operation. Perhaps our reluctance is due to the natural conservatism of our bridge designers who have even been slow in accepting long span steel girders, but more probably the patented features of this design and the license requirements are a retardant to its use.

Two notable examples of Dr. Leonhardt's continuous design are the Railroad Bridge at Heilbrann and the Highway Bridge near Heilbrann over the Neckar Canal.

The most famous of Dr. Leonhardt's designs is the 3-span continuous bridge over the Neckar Canal. It has 2 spans at 62' and a center span of 312' with a depth of only 5'-7" at the center. It is said to be the longest concrete girder span in the world. One of the continuous bridges built in this country is the grade separation structure at Waugh and Memorial Drive in Houston, Texas. This structure, on a sharp curve, has 2 spans at 45', 1 at 65' and 1 at 70' with an over-all length of about 229'. The entrie structure is banked. It has a 24" thick slab containing 58 stressing units in the section with economy holes between each unit.

Other plans prepared include the Little Falls Bridge in Maryland for the Corps of Engineers. This structure is similar to the Neckar Canal Bridge in that it has a center span of 216' and cantilevered counterweighted spans of about 30'. It is a box girder section 7' deep at the crown, and 12' deep at the support containing only 2 stressing elements.

A study has also been made for a continuous bridge in Providence over the Providence River having 10 spans with spans varying from 100 to 120' in length. This, too, is a box girder type.

California

The California Bridge Department has been one of the pioneers in concrete box girder designs. Many of their grade seperation bridges (conventionally reinforced) are so designed. These are generally of uniform depth for both simple and continuous spans and are readily adaptable to skews and almost any degree of curvature.

The Big Dalton Wash Bridge near El Monte, Figure 13, is of this type but is prestressed. It has a span of 115' and was designed for H20 and S16 loading. The depth of structure is 7'-3", the top slab is 6-¾" thick, the bottom slab is 5-½" thick and the interior walls are 8" thick. The stressing elements are in the bottom slab and walls.

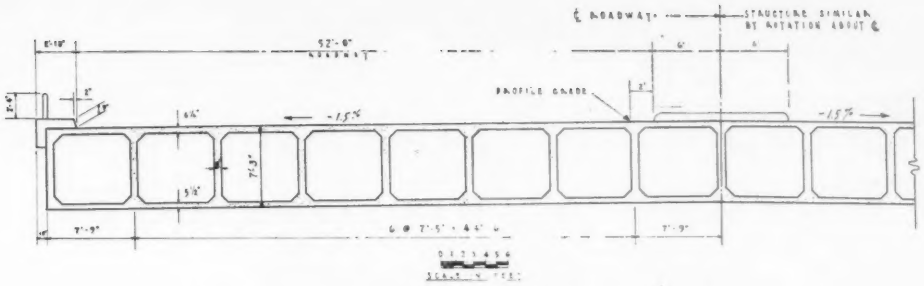


FIG. 13. Half Cross Section—Big Dalton Wash Bridge.

CONCLUSION

It has been an interesting and stimulating experience to have had some part in the development of prestressed concrete, and to have had an opportunity to follow the trends and development of prestressing in this country. In its early stages, it was not comparable in price with steel stringers, but its continued use and the adoption of more economical methods and the discarding of unnecessary features have brought it to the point of being highly competitive and desirable.

Pretensioning is rapidly gaining favor and I believe that this or a combination of pretensioning and post-tensioning will be the logical and economical solution in mass production. I would expect on moderate spans that post-tensioning will, in time, be used only on isolated structures in areas where pretensioning beds have not been set up and on the longer beams which, because of their extreme length and weight, will have to be produced and stressed at the bridge site.

What of the future of prestressing? I believe any good and economical design has a bright future in this country. We are in the midst of a gigantic highway program, the end of which cannot now be seen. Barring a national emergency, the program cannot be halted until the job is done, and that job includes the building of thousands of structures unthought of a few short years ago.

This is a great steel producing country and unless there is a shortage of critical materials, steel will not be replaced in bridge work by prestressing, but prestressing will probably be used far beyond our wildest dreams to help build the highways for an age when our future economy and, perhaps, our eternal existence will depend on our being a highly mobile country.

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Graphs for preliminary design of prestressed concrete beams . . .

By Paul Zung-Teh Zia

Assistant Professor
College of Engineering
University of Florida

Introduction

The design and analysis of a prestressed concrete beam often involves a tedious trial-and-error process. Unless experienced in this type of work, the structural designer faces a laborious task in selecting a proper trial section. For a preliminary design, such consumption of time cannot always be justified. At this stage, the designer is most interested in two problems, namely the probable depth of the member and its comparative cost. Although several "rules of thumb" have been suggested – such as span-depth ratio, percentage of corresponding reinforced concrete section, or an empirical formula, these rules do not readily lead to the solution of the problem without some additional computation. It is the purpose of this article to present a series of curves for direct solution based upon the simple concept of "kern" and stress distribution on a section.

Fundamental Concept

Consider a beam of arbitrary cross section as shown in Fig. 1 in which C_t & C_b are the distances of the extreme fibers from the neutral axis; K_t & K_b are the distances of the kern points from the neutral axis. From mechanics it is found that $K_t = \frac{r^2}{C_b}$, $K_b = \frac{r^2}{C_t}$ where $r^2 = \frac{I}{A}$

If this section is subjected to triangular stress distributions as shown in Fig. 2, the resultant "P" of these stresses is known to act at the kern points. If the allowable compressive stress in a prestressed concrete beam is fC and the allowable tensile stress is zero, the resultant of the compressive stress in the concrete, due to prestressing force alone, should be equal and opposite to the pull in the prestressing tendons as shown in Fig. 3. However, if the beam still maintains the triangular stress distribution, as in Fig. 2 (a), under the combined action of prestressing

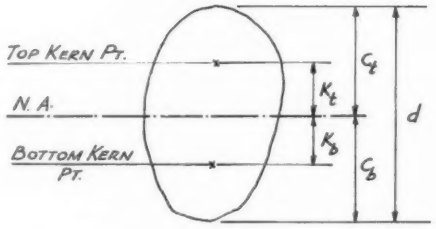


Figure 1

force and dead load moment of the beam, the prestressing tendons must be at a distance "a" from the lower kern point in order that the couple "Pa" balances the dead load moment as shown in Fig. 4. Therefore

$$M_{D.L.} = Pa \quad - - - - - - - - - - \quad (1)$$

In practice, this is possible provided that the distance "a" is reasonably small so that

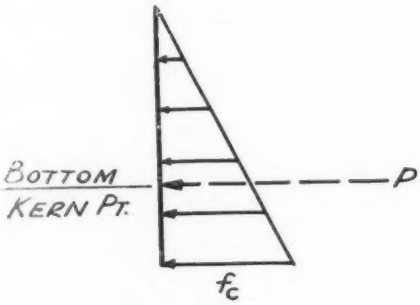


Figure 2 (a)

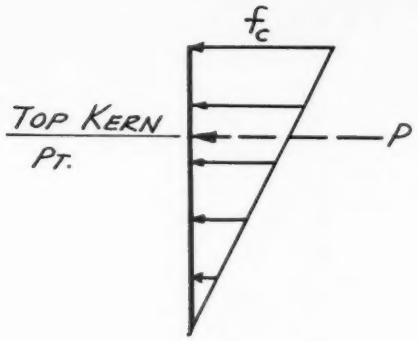
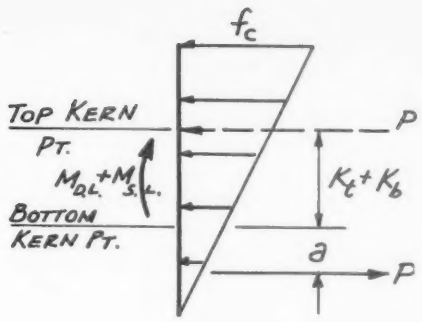


Figure 2 (b)



the prestressing tendons remain within the beam section. Upon the application of live load on the beam, the stress distribution will be as shown in Fig. 5. The internal couple P $(K_t + K_b + a)$ will be in equilibrium with the external moments $M_{D.L.} + M_{S.L.}$ Therefore $M_{D.L.} + M_{S.L.} = P\ (K_t + K_b + a) =$

$P\ (K_t + K_b) + P$ - - - - - - - - (2)

From Eqs. (1) and (2), $M_{S.L.} = P$

$(K_t + K_b)$ - - - - - - - - - - - - (3)

Thus by determining "P" and kern distances of a particular section with an assumed stress distribution, the resisting moment of the section against superimposed load can be found easily from Eq. (3).

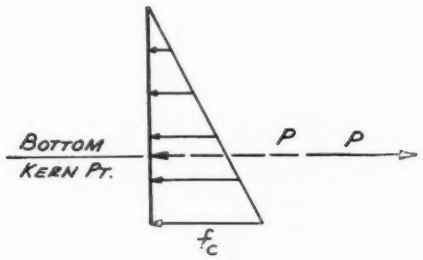


Figure 3

Preparation of Curves

In the following discussion, it is assumed that: (a) allowable compressive stress $f_c = 2000$ psi and allowable tensile stress in concrete = 0; (b) P is the effective prestressing force, i.e., loss of prestress has taken place; (c) the prestressing tendons will be curved or draped in the beam so that the stress distribution at mid-span governs the design.

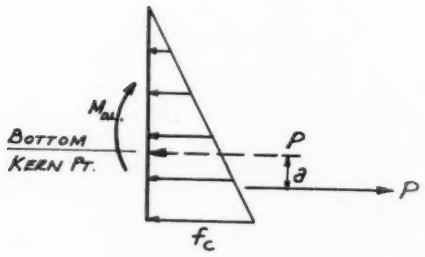


Figure 4

Case I. Modified I–Section, Depth 30" to 60"

In prestressed concrete design, one of the first difficulties which confronts the structural designer is the proportioning of the section. Although there have been several theoretical treatments on this subject, the actual shape of a beam section still depends largely upon practical considerations. For example, the web thickness of a beam should provide sufficient coverage for the steel and also ample room for the proper placing of concrete into the bottom flange. For a beam with depth "d" between 30 in. and 60 in., a web thickness of 0.1d seems to be adequate.

The optimum thickness of the bottom flange is usually 0.15d. The width of the bottom flange can be assumed to be one-half that of the top flange. Since the top flange is wider, its thickness is, therefore, assumed to be slightly less than that of the bottom flange. Thus a modified I-section is obtained as shown in Fig. 6 with every dimension expressed in terms of, and as a fraction, of the depth "d." The properties of this section determined by the usual methods are given below:

$$A = (0.175\,\alpha + 0.075)\,d^2 \qquad (4)$$

$$C_b = \bar{Y} = \frac{0.101\,\alpha + 0.039}{0.175\,\alpha + 0.075}\,d$$

$$C_t = d - C_b = \frac{0.074\,\alpha + 0.036}{0.175\,\alpha + 0.075}\,d$$

$$I = (0.0323\,\alpha + 0.0045)\,d^4$$

$$r^2 = \frac{(0.0323\,\alpha + 0.0045)}{(0.175\,\alpha + 0.075)}\,d^2$$

$$K_t = \frac{r^2}{C_b} = \frac{0.0323\,\alpha + 0.0045}{0.101\,\alpha + 0.039}\,d$$

$$K_b = \frac{r^2}{C_t} = \frac{0.0323\,\alpha + 0.0045}{0.074 \quad + 0.036}\,d$$

$$K_t + K_b = \frac{(0.0323\,\alpha \quad 0.0045)}{(0.101\,\alpha \quad 0.039)} \times \frac{(0.175\,\alpha \quad 0.075)}{(0.074\,\alpha \quad 0.036)}\,d \qquad (5)$$

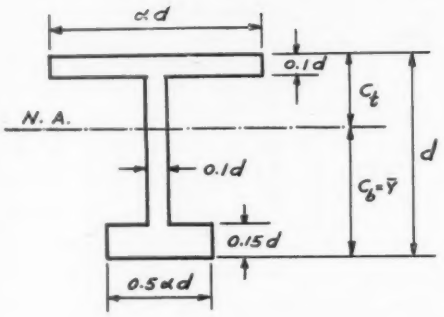


Figure 6

In order to obtain rapid solutions of equations (4), (6) and (7), a series of curves are plotted for different values of α in Plates 1, 2 and 3. When the moment due to superimposed load is given, the depth of the beam can be selected from Plate 1. For any selected depth of beam, the area of concrete and the prestressing force required can be determined from Plates 2 and 3, which in turn determine the cost of the selected section.

When this section is under dead load and prestressing force, the resultant of the stress block, having 2000 psi bottom fiber stress and zero top fiber stress, will be:

$$P_1 = (148.75\,\alpha + 71.25)\,d^2$$

Under full working load, the resultant of the stress block, having zero bottom fiber stress and 2000 psi top fiber stress, is:

$$P_2 = (201.25\,\alpha + 78.75)\,d^2$$

It is obvious that the prestressing force "P" should equal to the smaller of these two values. Hence:

$$P = P_1 = (148.75\,\alpha + 71.25)\,d^2 \qquad (6)$$

Substituting Eqs. (5) and (6) into Eq. (3):

$$M_{S.L.} = P\,(K_t + K_b) = (148.75\,\alpha + 71.25) \frac{(0.0323\,\alpha + 0.0045)}{(0.101\,\alpha + 0.039)} \times \frac{(0.175\,\alpha + 0.075)}{(0.074\,\alpha + 0.036)}\,d^3 \qquad (7)$$

CASE II. Symmetrical I-section, Depth 30" to 60"

As in Case I, the same treatment is given to a symmetrical I-section shown in Fig. 7. The properties of this section are listed below:

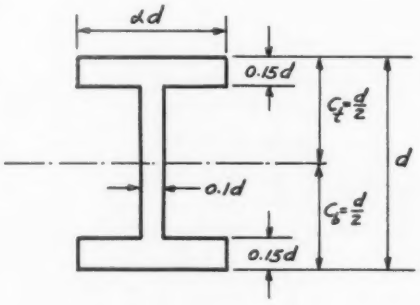


Figure 7

$$A = (0.3\alpha + 0.07)\, d^2 \qquad (8)$$

$$C_b = C_t = \bar{Y} = \frac{d}{2}$$

$$I = \frac{(0.657\alpha + 0.0343)}{12}\, d^4$$

$$r^2 = \frac{I}{A} = \frac{(0.657\alpha + 0.0343)}{12(0.3\alpha + 0.07)}\, d^2$$

$$K_t = K_b = \frac{r^2}{C_b} = \frac{2r^2}{d}$$

$$K_t = K_b = \frac{4r^2}{d} = \frac{(0.657\alpha + 0.0343)}{3(0.3\alpha + 0.07)}\, d$$

and $$P = (300\alpha + 70)\, d^2 \qquad (9)$$

Again from Eq. (3)

$$M_{S.L.} = P(K_t + K_b) =$$

$$\frac{1000(0.657\alpha + 0.0343)}{3}\, d^3 \qquad (10)$$

Curves for equations (8), (9) and (10) are plotted in Plates 4 and 5.

Example

In order to illustrate the use of these curves, let us consider the design of a 60 ft. modified I-beam resisting a superimposed moment of 800 ft. - kips. From Plate 1, five different sections of the modified I-beam with depths ranging from 44½ in. to 53¼ in. can be selected. If the use of the 48¼ in. beam is preferred, one can ascertain, from Plates 2 and 3, the concrete area and prestressing force required for this section to be 425 sq. in. and 375 kips, respectively. Now the weight of this beam would be:

$$\frac{425}{144} \times 150 = 443 \text{ lb. per ft.}$$

and

$$M_{D.L.} = \frac{WL^2}{8} = 443\ \frac{(60)^2}{8} =$$

199 ft. - K = 2400 in. - K

From Eq. (1) and Fig. 4:

$$\text{"a"} = \frac{M_{D.L.}}{P} = \frac{2400}{375} = 6.4 \text{ in.}$$

This value of "a" is only 13.2 percent of the depth of beam and the prestressing tendons can easily be placed within the concrete section. If "a" is considerably larger so that the prestressing tendons cannot be properly arranged within the concrete section, the next deeper section should be considered.

Since the trial beam has a cross-sectional area of 425 sq. in., the amount of concrete required is 0.109 cu. yd. per ft. If ⅞ in. 7-wire strands are used, then the amount of prestressing steel required is $\frac{375}{11.2} = 34$ strands per beam, where the working load pull for a ⅞ in. strand is 11.2 kips. Knowing the quantities of both major materials, and by assuming a reasonable unit cost for each, the designer will be able to make the proper cost study.

Discussion

This article demonstrates the method of preparing such useful graphs for preliminary design of prestressed concrete modified-I and I-beams. Similar graphs can be prepared to include all common types of prestressed concrete beam sections. The graphs in this article can be extended to include beams less than 30 in. deep in which case the web thickness co-efficient should be increased from 0.1 to 0.15 in order to provide the practical minimum web thickness. Since $M_{S.L.}$ and P are directly proportional to the allowable stress f_c, the resisting moment and prestressing force for values of f_c other than 2000 psi can be obtained by multiplying the results from these curves by $\frac{f_c}{2000}$.

It is also interesting to observe from these graphs that, for a given resisting moment, beams of different depths can be selected. These beams, having a balance between their depths and flange widths, will have almost the same cross-sectional area and consequently require almost the same amount of prestressing force.

Acknowledgment

The author wishes to express his appreciation to Professor Ralph W. Kluge and Doctor A. M. Ozell of the Department of Civil Engineering, University of Florida, for their valuable criticism and suggestions.

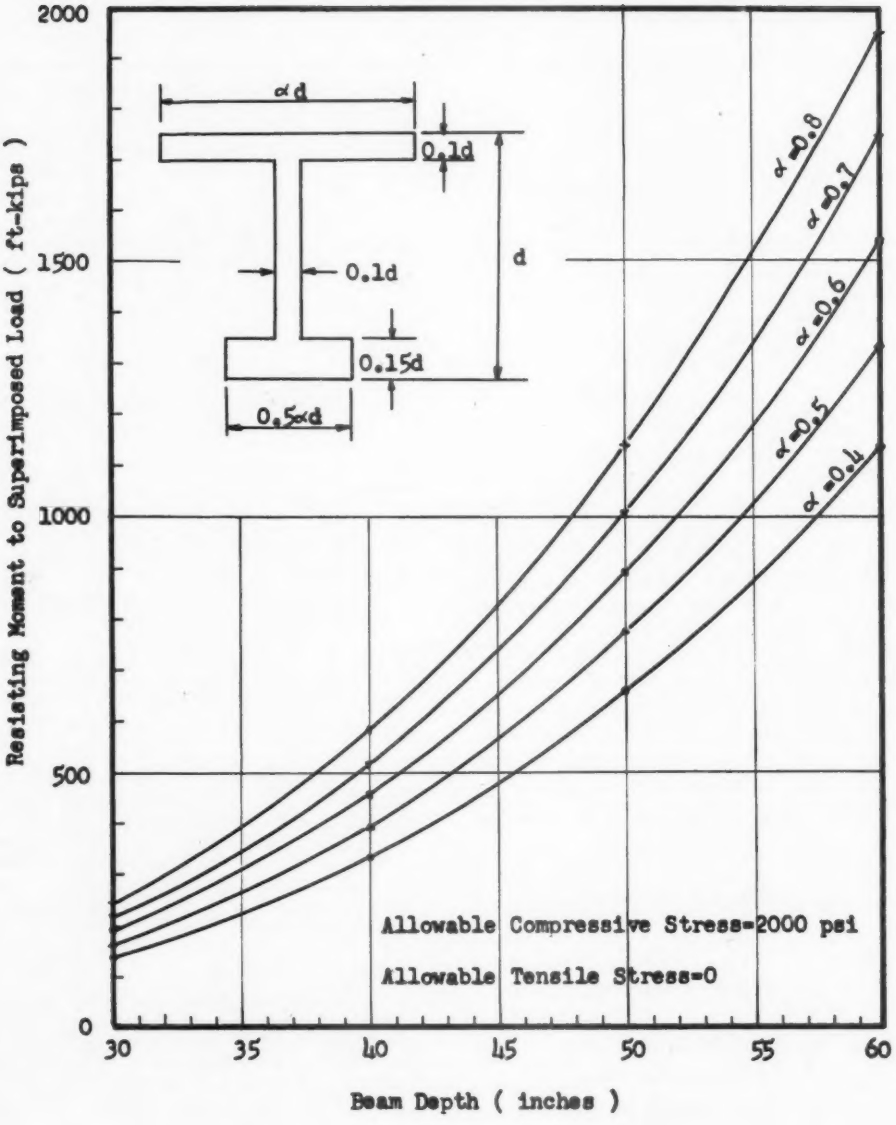


Plate 1 Resisting Moment vs Beam Depth - Modified I-section

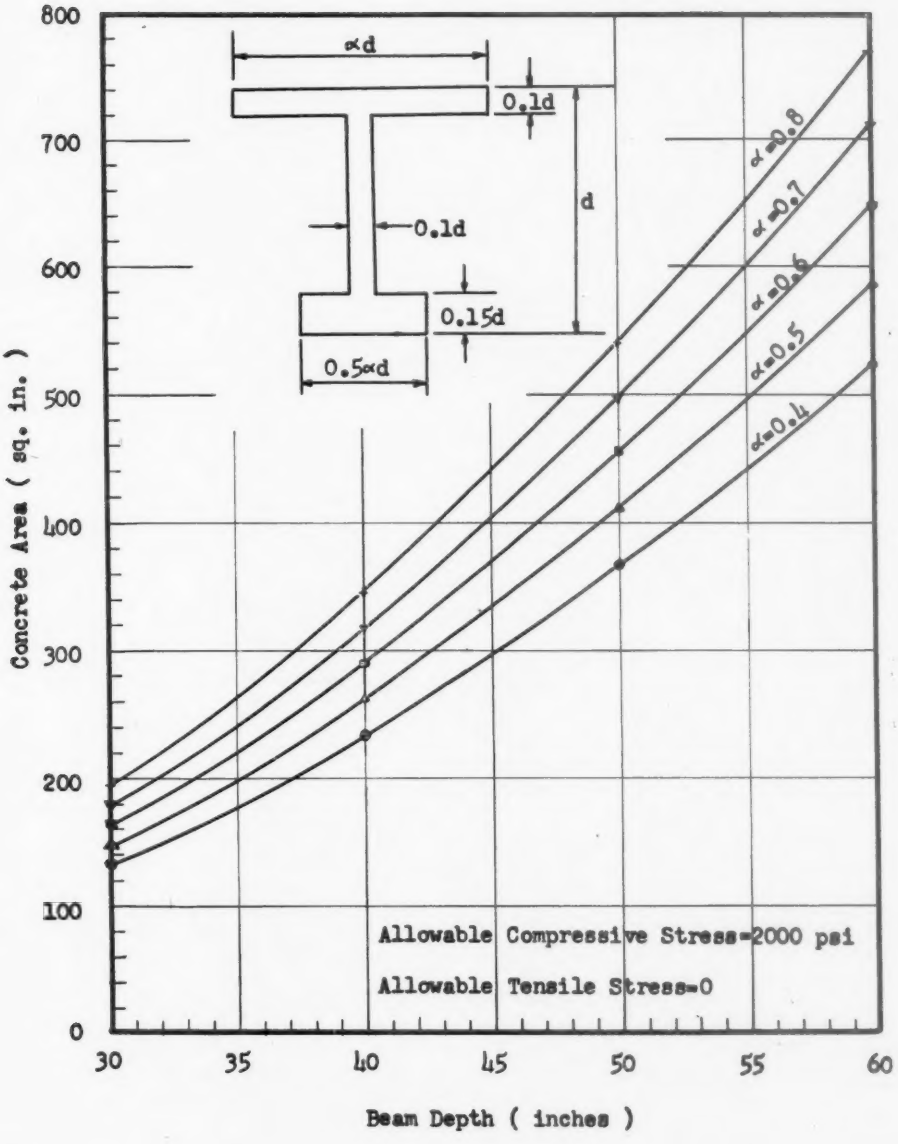


Plate 2 Concrete Area vs Beam Depth - Modified I-section

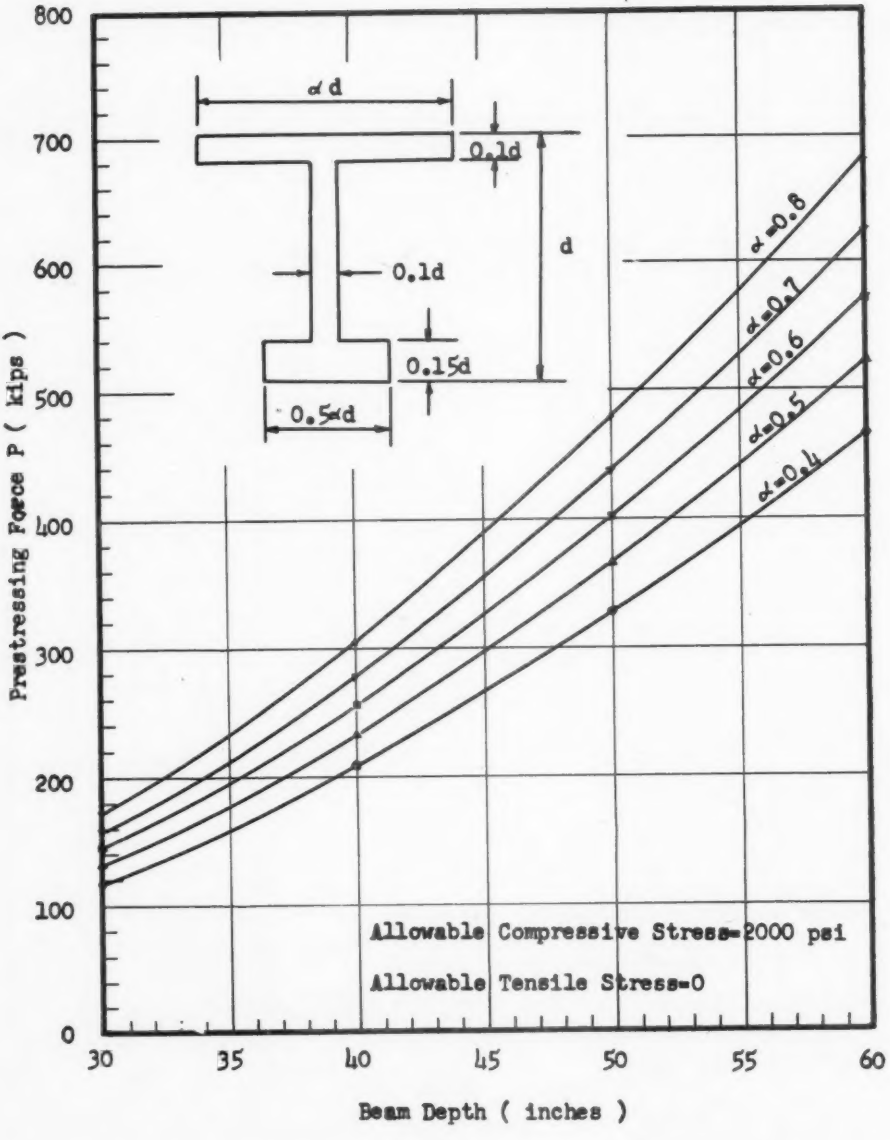


Plate 3 Prestressing Force vs Beam Depth - Modified I-section

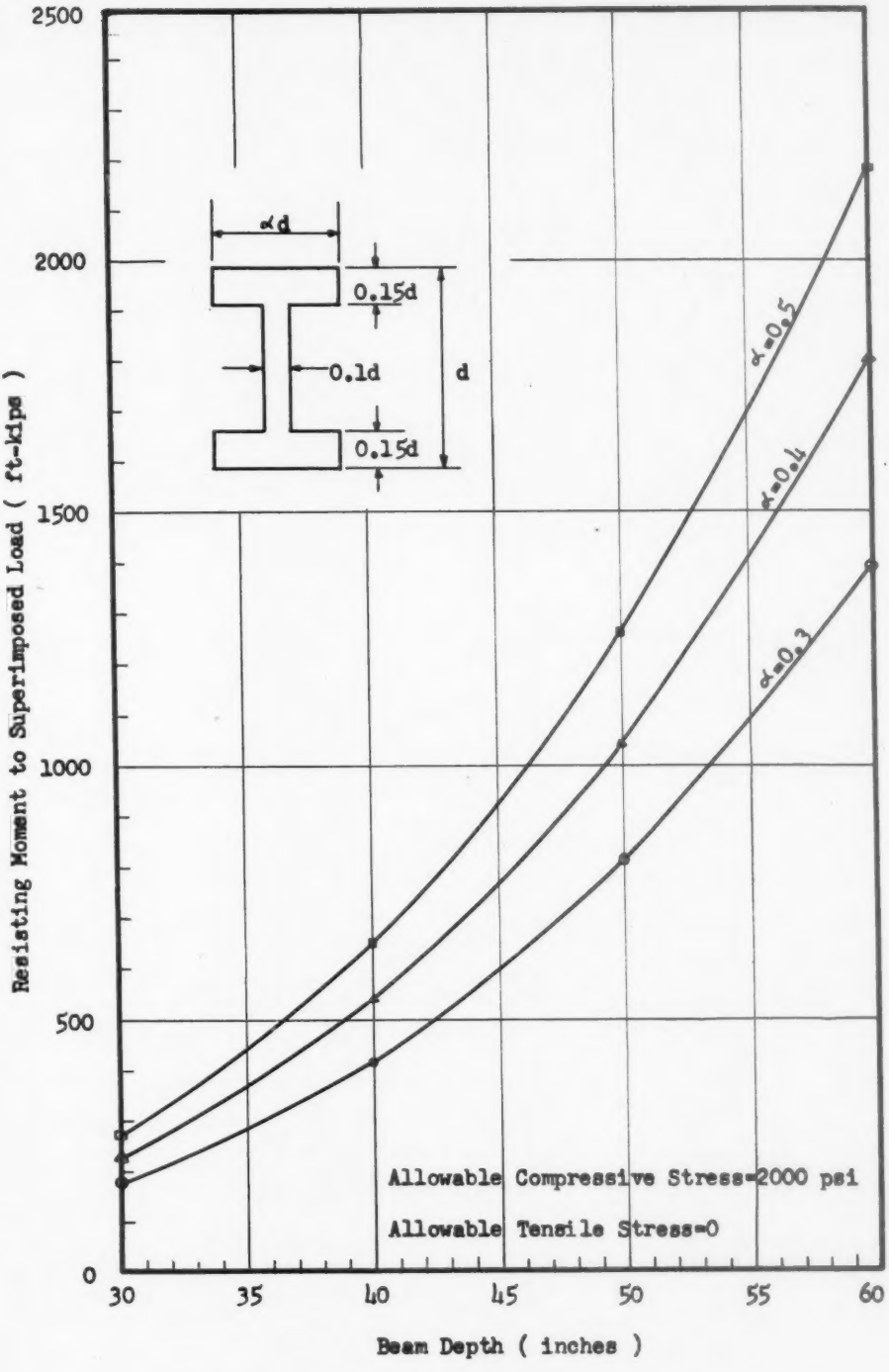


Plate 4 Resisting Moment vs Beam Depth - I section

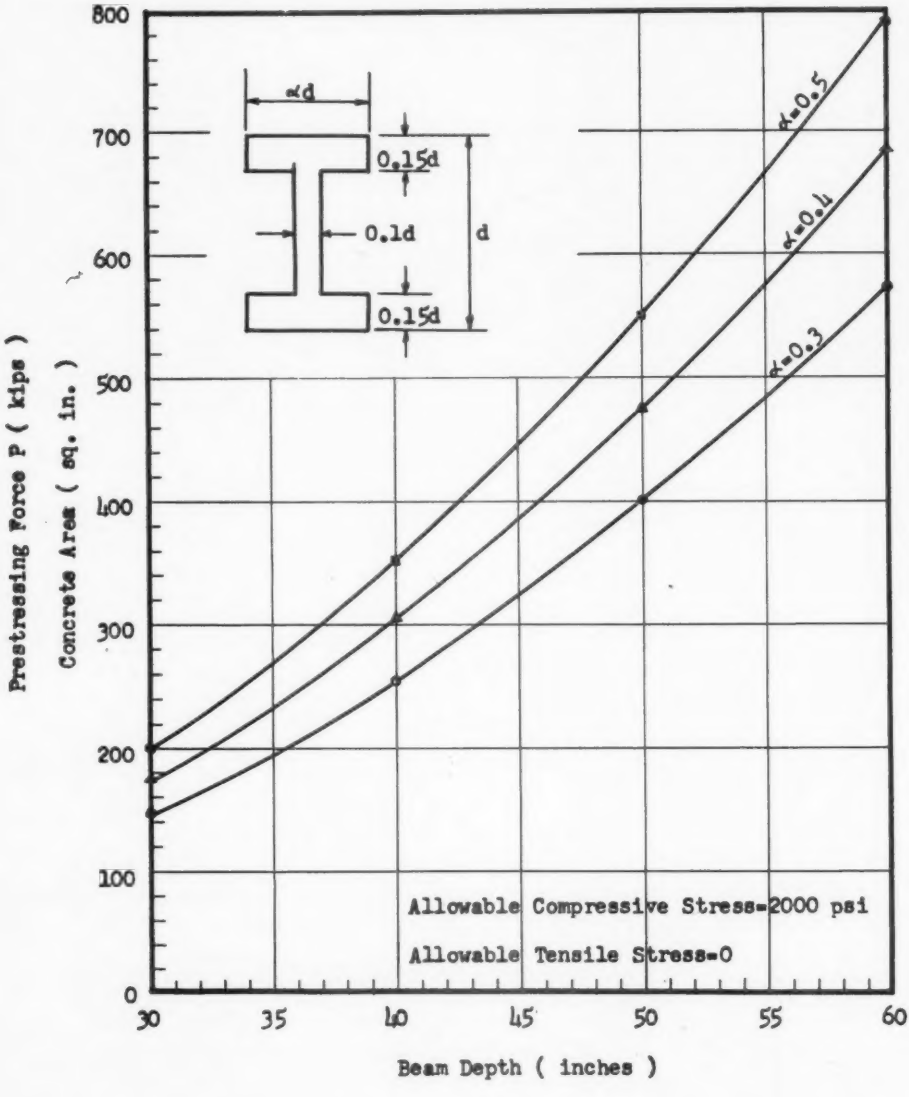


Plate 5 Concrete Area and Prestressing Force vs Beam Depth - I section

The Start of the Lake Pontchartrain Bridge

7-WIRE S. R. STRAND CROSSES

Lake Pontchartrain

By R. O. KASTEN, *Director of Research, Union Wire Rope Corp.*

The longest vehicular bridge in the world is now nearing completion across Lake Pontchartrain in the greater New Orleans area. For 24 miles simple beams 56 ft. long and 33 ft. wide pre-tensioned prestressed with ⅜ in. wire strand are supported on hollow 54 in. post-tensioned pilings. With approximately 2250 roadway sections needed, the contractor, The Louisiana Bridge Company, applied all the production techniques and time-saving procedures to a central casting yard for pre-tensioning. These roadway slabs, a hefty 200 tons, are moved by gantry cranes to barges, towed to their respective piers and dropped in place by a large barge mounted crane. Here is another case where the productive geniuses in America have used pre-tensioned prestressed concrete to build a lower cost bridge, in less time and with lower maintenance than with any of the other competitive building materials.

Prestressed concrete is not new. Although patents were granted for prestressing to an American engineer as early as 1886, first attempts at prestressing were not successful. The early pioneers did not foresee the loss in prestressing force due to elastic shortening, bending of concrete, creep of concrete, shrinkage of concrete and creep of prestressing tendon. The average aggregate loss in prestressing force may run as high as 18 per cent. Consequently, when mild steel bars were used as the stressing tendon, the prestressing loss was sometimes as great as the total prestressing force originally applied, thus, prestressed concrete turned out to be reinforced.

Selected hot rolled high carbon steel rods of suitable diameters with a chemical analysis of

Carbon	0.75 – 0.85%
Manganese	0.45 – 0.85%
Phosphorous	– 0.045% Maximum
Sulphur	– 0.050% Maximum

are normalized (a process called patenting), cleaned and cold drawn a sufficient amount to produce the required final physical properties. With the ultimate strength and yield strength so high, this strand is able to absorb the loss in prestressing forces and still

have sufficient strength to hold the concrete in compression. The ⅜ in. wire strand (6 wires helically laid around a king wire), furnished The Louisiana Bridge Company for the Pontchartrain Bridge, has the following properties:

Minimum Metallic Area	.0799 Sq. In.
Minimum Ultimate Tensile Strength	20,500 lbs.
Minimum Yield Strength .2% Offset Method	16,500 lbs.
Average Modulus of Elasticity	28,000,000 PSI
Minimum Ultimate Elongation	3% in 10"
Length of Lay (Approx.)	4½" to 5"

These specifications were determined by the design engineers, Palmer and Baker, Mobile, Alabama, and are slightly higher than the present standards. The minimum tensile strength is equivalent to 257,000 lbs. per square inch, several times the strength of ordinary mild reinforcing steel. Figure 1 is a typical stress-strain curve of the ⅜ in. Tufwire strand. The shape of this curve displays a high yield strength coupled with a substantial, but not excessive ultimate elongation at rupture. The minimum yield strength .2% offset should equal at least 80 per cent of the ultimate tensile strength. The wire should be capable of a reasonable amount of cold deformation without fracturing for ductility requirements.

The last step in fabricating wire strand or wire is stress relieving. Here the tendons are pulled through a lead bath with temperatures of 750° to 800°F. for a few seconds. This process does produce changes in the physical properties. For instance, there is a notable increase in ultimate elongation. This time temperature treatment removes some of the residual stresses making the strand much easier to handle at the job site. Any oil, grease or foreign material is removed from the strand to allow better bonding of concrete. Stress relieving reduces the loss due to load relaxation up to 70 per cent of the ultimate strength and affords more uniform physical properties.

Stress corrosion has received considerable emphasis recently. Tests show cold drawn stress relieved wire to be far superior to oil tempered wire.

Wire strand is available in 1/4 in., 5/16 in., 7/16 in. and 1/2 in. diameters. Table 1 shows the minimum requirements for these sizes.

Strand can be furnished either in coils or reels. Coils are 5 ft. in diameter and can be furnished in a maximum continuous length of 3500 ft. in the 3/8 in. diameter. Strand in coils is generally easier to handle. It will "pay off" straighter and is cheaper since the consumer does not have to pay freight on the dead weight of the reel.

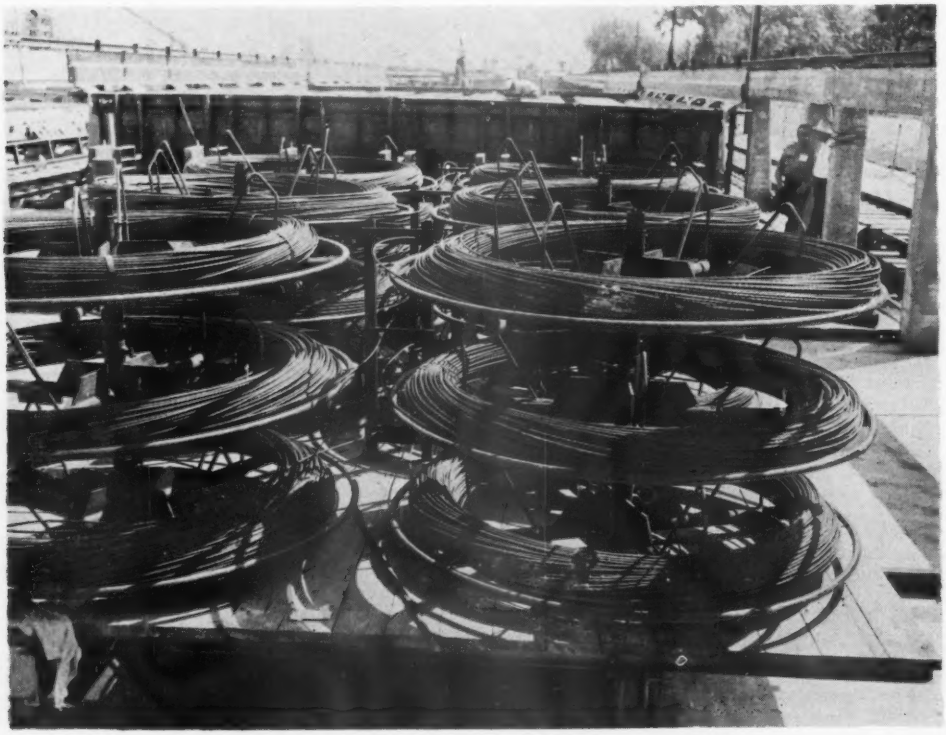


PHOTO 2. Swift car fully loaded. Louisiana Bridge Company.

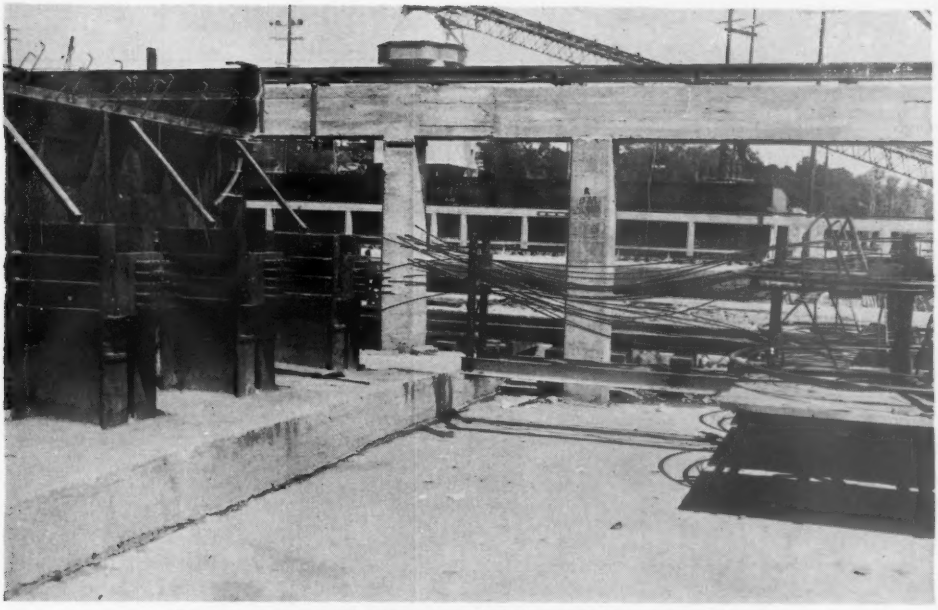


PHOTO 3. Strand has been pulled through last tee and cut.

Photograph 2 shows the swift car used by The Louisiana Bridge Company which has a capacity of 25 coils and is fully loaded. Photograph 3 shows the strand cut but still extending through the guide plate, while Photograph 4 shows the swift car empty and ready to be reloaded. The coils were shipped in lengths of 3388 ft. to avoid any remnants longer than a few feet.

From the Photographs 2, 3 and 4 one can see how all 25 strands "pay off" through guide plates in the middle of the swift car and require only a minimum amount of space. For this job, reels would have been bulky, besides requiring a stronger swift car to support the extra weight.

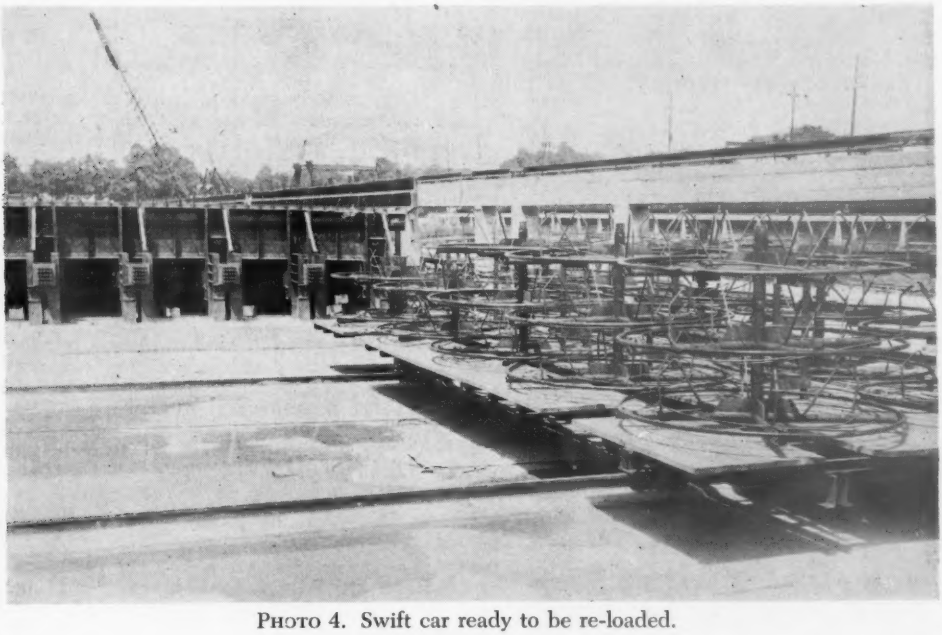


PHOTO 4. Swift car ready to be re-loaded.

TABLE 1

TUFWIRE STRAND

Nominal Diameter Inches	Weight Per 1000 Feet Pounds	Minimum Area Square Inch	Minimum Ultimate Strength Pounds
1/4	122	.0356	9,000
5/16	210	.0578	14,500
7/16	386	.1089	27,000
1/2	521	.1438	36,000

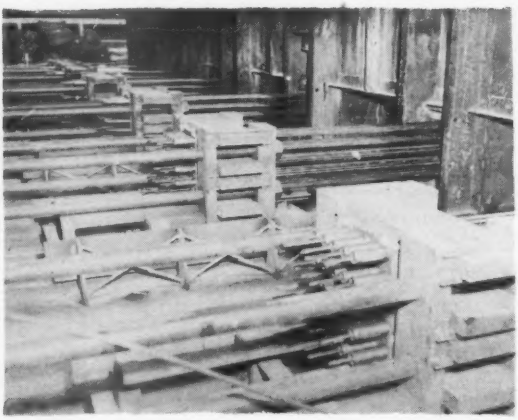


PHOTO 5. Jacks tensioning strand.

Clamps were used to hold the strand at the jacking end as well as at the front end. Freyssinet Company, Inc., was employed by the contractor as consultants for the pre-tensioning bed including the cleverly designed jacking arrangement. Photograph 5 shows the strand under tension at the jacking end while Photograph 6 shows the strand secured with clamps after tensioning at the front end.

High tensile wire and strand is taking its place in making the phenomenal building material, prestressed concrete practical. With our American production minded and cost conscious technicians, prestressed pre-tensioned concrete will eventually mean more and better bridges, buildings and structures of various types for everyone.

PHOTO 6. Strand securely anchored with Reliable clamps . . . front end.

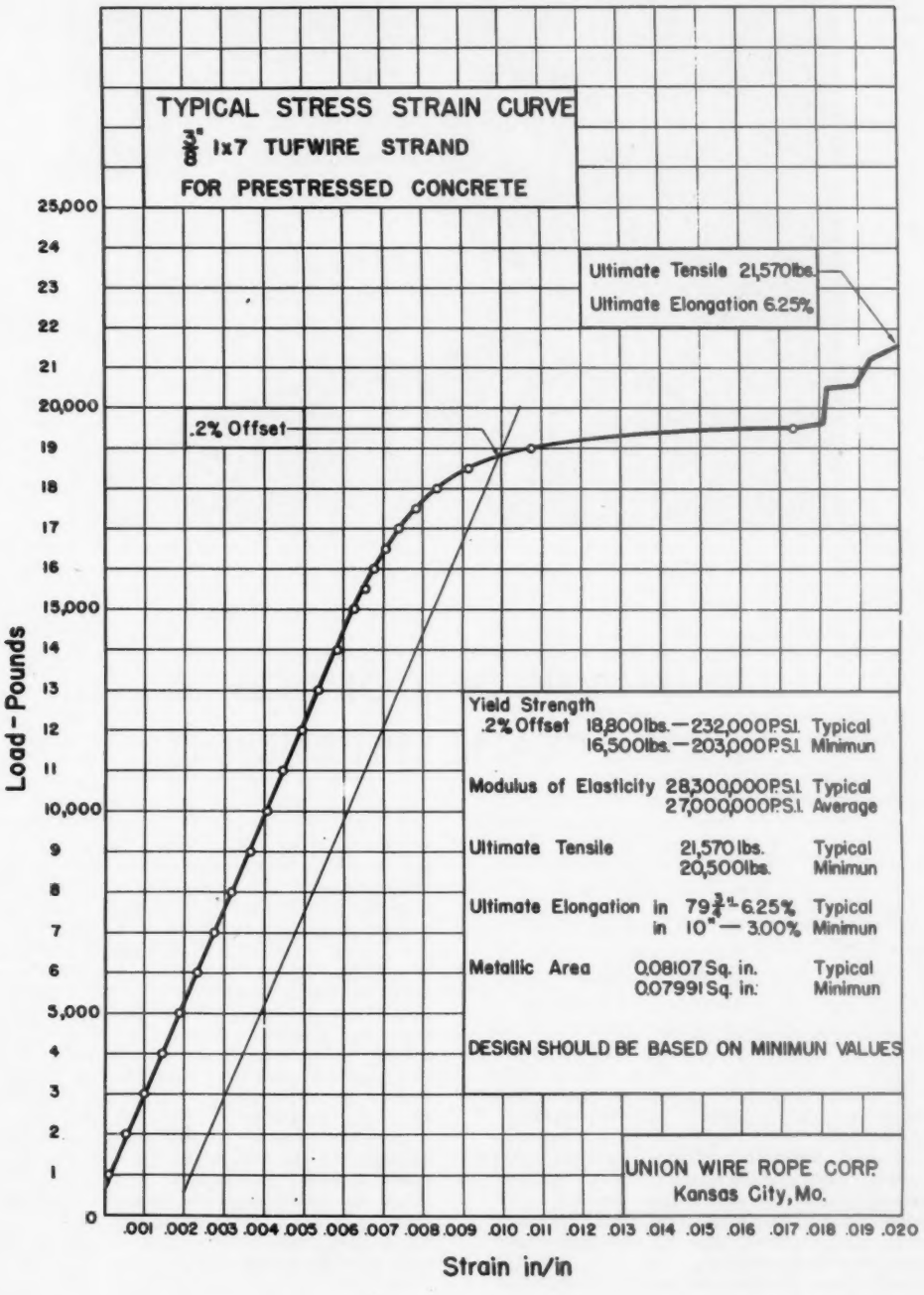

TYPICAL STRESS STRAIN CURVE
3/8" 1x7 TUFWIRE STRAND
FOR PRESTRESSED CONCRETE
Ultimate Tensile 21,570lbs.
Ultimate Elongation 6.25%
.2% Offset
Load - Pounds
25,000
24
23
22
21
20,000
19
18
17
16
15,000
14
13
12
11
10,000
9
8
7
6
5,000
4
3
2
1
0
.001 .002 .003 .004 .005 .006 .007 .008 .009 .010 .011 .012 .013 .014 .015 .016 .017 .018 .019 .020
Strain in/in
Yield Strength
.2% Offset 18,800lbs. — 232,000 P.S.I. Typical
16,500lbs. — 203,000 P.S.I. Minimun
Modulus of Elasticity 28,300,000 P.S.I. Typical
27,000,000 P.S.I. Average
Ultimate Tensile 21,570 lbs. Typical
20,500lbs. Minimun
Ultimate Elongation in 79 3/4" 6.25% Typical
in 10" — 3.00% Minimun
Metallic Area 0.08107 Sq. in. Typical
0.07991 Sq. in. Minimun
DESIGN SHOULD BE BASED ON MINIMUN VALUES
UNION WIRE ROPE CORP.
Kansas City, Mo.

Figure 1

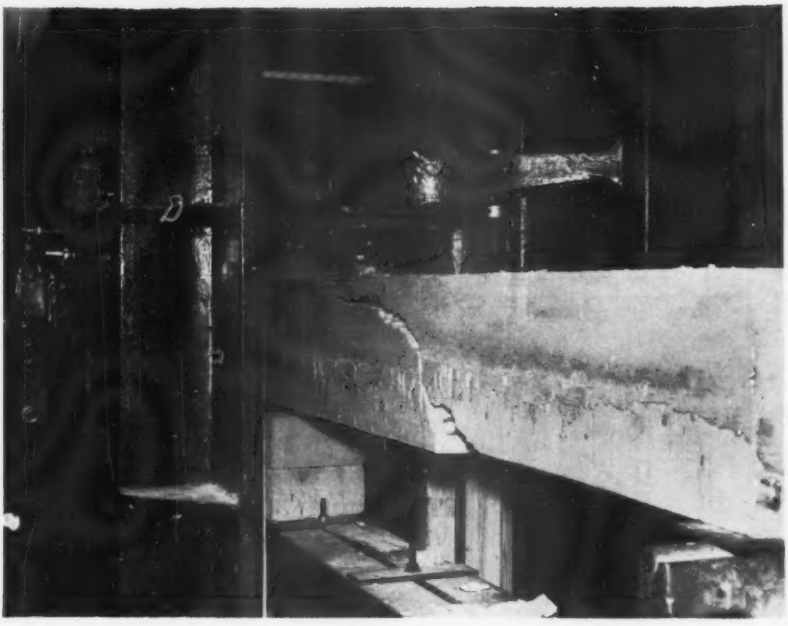


FIG. 2. Typical failure of Composite Beam.

BEHAVIOR OF COMPOSITE LINTEL BEAMS IN BENDING

By

DR. A. M. OZELL,

J. W. COCHRANE

College of Engineering
University of Florida
Gainesville, Florida

In many phases of prestressed concrete construction, composite beams contribute substantially to economical and desirable structures. Savings resulting from mass production, easy handling, and good quality control make the prestressed precast units exceedingly advantageous both for light and heavy construction. The cast-in-place concrete core (web or flange or both), poured later to form an integral part of the precast unit, makes it possible for the composite section to have the desired dimensions and proper finish, and results in a structural member which can carry much heavier loads than the precast unit.

The usual procedure in the design of composite beams is to assume that the cast-in-place concrete acts integrally with the precast unit and that the resulting beam behaves as though it were monolithic; i.e., assumptions made for the rational design procedure involving elastic behavior are extended to include this type of construction. Consequently, in the design of such members several questions arise: Is there a limit to the size of the core in relation to the size of the bare lintel-beam beyond which elastic behavior is no longer possible? Does the geometry of the bare lintel-beam or the cast-in-place portion affect the elastic behavior of the composite action? To what extent should the two parts of the composite beam be tied together to prevent the tendency to vertical and horizontal separation? Does the natural bond of the concrete at the joint provide reliable resistance to horizontal shear? To what extent does differential shrinkage affect the strength of composite beams Would repeated loads have damaging effects on composite construction?

Several tests have been conducted to find answers to some of these questions (See

References 1, 2, 3). The tests covered in this report were directed to answer the following:

1. Does the relative size of the cast-in-place portion of a composite beam with respect to the size of the prestressed bare beam affect elastic behavior?
2. Does the natural bond of the concrete at the joint provide reliable resistance to horizontal shear?

Description of Specimens

In order to determine the size effect of the cast-in-place portion with respect to the size of the bare prestressed lintel-beam, eleven specimens* having the cross sections shown in Fig. 1 were tested. Two specimens were bare lintel-beams (with no core), and three specimens in each group had D/d (the ratio of the core depth to the depth of the bare lintel-beam) of 1.10, 1.56 and 2.07. In these composite beams mechanical ties, generally used to insure the bonding of the two surfaces or to prevent any slip due to horizontal shear, were omitted. Furthermore, the surface of the bare lintel-beam in contact with the core was extremely smooth.

The lintels. in which Type III cement was used, were wet burlap cured for four days before stress release at which time cylinder tests indicated strengths in excess of 4000 psi. The cores, in which Type I cement was used, were placed on the lintels, vibrated and cured under wet burlap for 7 days, then dry cured until testing. During pouring and curing of the core the lintels were supported at 2 ft. 6 in. from each end.

Test Procedure

The specimens were loaded with a single concentrated load placed at mid-span. Using mechanical gages strains were measured on the side, top and bottom surfaces of the beam. Deflection measurements were also taken. Cylinder tests were made to determine the strength of concrete used in the lintels and in the core. The 6 x 12 in. cylinders were cured under the same conditions and about the same length of time as the corresponding parts of the composite beams.

These tests did not include a study of differential shrinkage since it was assumed that for such cross sections its effect would be negligible.

Test Results

Table I presents a summary of test results. The bare lintels cracked at an average load of 4500 lbs. and failed in compression at an average load of 4935 lbs. Composite beams with cast-in-place cores having D/d ratio of 1.10 cracked at an average load of 18,050 lbs. and failed in shear at 26,300 lb. load.

The cracking and failure loads for composite beams with 1.56 D/d ratio were 29,250 lbs. and 44,300 lbs., respectively; and for composite beams with 2.07 D/d ratio the values were 33,300 lbs. and 54,300 lbs., respectively. Fig. 2 shows the type of failure which occurred in the composite beams with 1.10 D/d ratio.

Fig. 3 shows the strain distribution in the composite beams having various D/d ratios. The measured strain distribution was almost linear up to nearly the cracking load which substantiates the elastic theory commonly used in the design of composite sections. The elastic behavior of these beams can be observed also in Fig. 4 which represents the relationship between superimposed moment vs. e/y (e measured compressive unit strain and y distance of the fiber from the calculated gravity axis) for beams with various D/d ratios. It should be noted that the bare lintels did not quite follow this pattern.

Fig. 5 presents the superimposed load vs. center deflection curves, Fig. 6 the calculated stress distributions at cracking loads and Fig. 7 the variation of the total moment vs. various D/d values both at cracking and at failure loads. In Figures 3 through 7 test values for beams No. 2, 4, 8 and 11 were presented as these were representative of the behavior of other beams in the respective series of D/d ratios.

Discussion of Test Results

The test results indicated that the natural bond and friction between the bare lintel and the core provide for sufficient horizontal shear resistance so that composite action is possible. Test conducted (see Ref. 3) on a composite Tee-Section bridge member without any keys, but with tie-down steel only, showed no differential sliding between the elements up to ultimate load.

None of the composite beams tested showed any slip between the two surfaces or vertical separation until after the cracking load was reached. This seems reasonable since the computed average bond stress between contact surfaces, determined by using the expression VQ/Ib, was in the order of 50 to 60 psi and not large enough to cause any distress.

During the tests initial cracks appeared at the bottom of the composite beams and generally near the mid-span. On beams with D/d ratio of 1.56 and 2.07 the cracks opened suddenly with a popping sound and immediately extended approximately to the centroidal axis. At cracking loads the calculated fiber stresses at the bottom of core were in the order of 957 psi to 1137 psi (see Fig. 6), large enough to cause rupture at the bottom of the core. However, for the core to rupture, the concrete had to undergo the necessary elongation. This was prevented, by bond between the two surfaces, since the lintel having much smaller tensile stress in its bottom fibers and considerable compression in its top fibers (due to prestressing) had not undergone sufficient elongation for this to happen. Hence, only at some critical point the transfer of tension from the core to the lintel was enough to

*The beam specimens were furnished by Stresscrete Co., Leesburg, Florida.

cause the premature cracking of the composite section. This is believed to be the reason for the popping sound (probably simultaneous breaking of bond between the two contact surfaces) and the immediate extension of the cracks up to the centroidal axis. That the bond between the contact surfaces was not materially damaged until just before cracking is also substantiated by the load-deflection curves of Fig. 5 which follow a straight line nearly up to the cracking loads.

The design of such members might be improved greatly by placing several reinforcing bars at the bottom of the core to strengthen the concrete against the apparent large tensile stresses.

At subsequent loads composite beams with D/d of 2.07 developed diagonal tension cracks on both sides of the center load and these cracks progressed upward at an incline, about 30 degrees with the horizontal. Just before failure these diagonal cracks joined to make an arc leaving less than 2 in. of concrete intact at the top of the beam.

In beams with D/d ratio of 1.10 the initial cracks were very small at first and extended about 4 in. from the bottom of the composite section. As the loads were increased, wide diagonal tension cracks opened suddenly with a popping sound and extended several feet in length. Fig. 2 shows a typical failure of these beams where plane of failure at the bottom coincided with the terminal of the two unbonded strands.

The ultimate failure of composite beams was by yielding of prestressing strands. The bare lintel-beams, on the other hand, failed in compression which was expected since the computed stress in the extreme fiber was about 4600 psi compression corresponding to about 0.84 f'c.

The moment values imposed on the composite sections are given in Table II. The total moment values at observed cracking load are given in column (2) and the calculated "design" moment values (based on zero tension in the lintel) are given in column (4). The apparent factor of safety against cracking for beams with D/d ratios of 1.10, 1.56 and 2.07 were 1.71, 1.64 and 1.41, respectively. Similarly, the ultimate moment values were 2.38, 2.44, 2.28 times greater than the "design" moment for D/d ratios of 1.10, 1.56 and 2.07, respectively. It should be noted that the cracking load capacity of this type of composite beams seems to reach an optimum value at D/d ratio of about 2.5 (this was extrapolated in Fig. 7).

The ultimate moment capacity of various beams was also calculated assuming a rectangular stress distribution, 0.85 f'c extreme fiber stress and "kd" values of 0.1d (the observed compression depths were about 0.1d). The curve for calculated moment values is given in Fig. 7 for a comparison with the test results.

From the superimposed moment vs. e/y relationships of Fig. 4 the resisting moments were calculated using as their value the slope of the M vs. e/y curves. These were in close agreement with the calculated resisting moment values (shown in parenthesis in Fig. 4) determined by using the expression $M = \frac{sI}{y} = \frac{eEI}{y}$.

Studies were also conducted to determine the degree of agreement between the theoretical moment of inertia of the section and the actual moment of inertia. Fig. 8 represents the theoretical moment of inertia curve for various depths of a rectangular beam. For comparison actual moment of inertia values, computed from the measured deflections and the tangent moduli of the various beams were plotted (in solid circles). Also, moment of inertia values calculated from the moment vs. e/y relationships of Fig. 4, expressed as $I = \frac{Mc}{s} = \frac{Mc}{eE} = \frac{My}{Ee}$, were plotted. It is interesting to note that the average moment of inertia values calculated in this manner substantially agree with theoretical values obtained by assuming rectangular cross section for the composite beams.

Conclusions

The following conclusions can be drawn from the results of tests described above:

1. The assumption of elastic behavior of composite beams of similar dimensions seems valid.
2. Natural bond between the bare beam and cast-in-place portion provide reliable resistance to horizontal shear.
3. The load carrying capacity of this type of composite beam appears to reach a maximum at D/d ratio of about 2.5.

References

1. Evans, R. H., and Parker, A. S., "Behavior of Prestressed Concrete Composite Beach," *ACI Journal*, May 1955, p. 861.
2. Revesz, Stephen, "Behavior of Composite T-Beams with Prestressed and Unprestressed Reinforcement," *ACI Journal*, Feb. 1953, p. 585.
3. Dean, W. E., and Ozell, A. M., "Static Testing of a Composite Prestressed Concrete Bridge Section Constructed without Shear Keys," *Engineering News Record*.

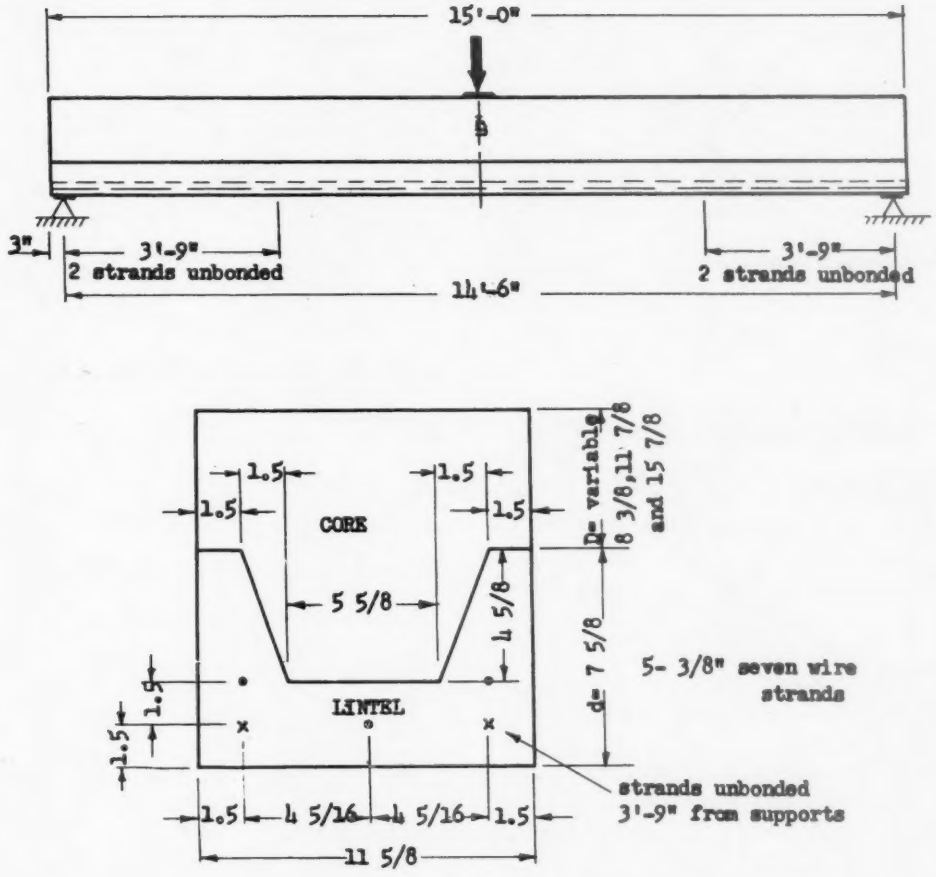


Fig. 1 Test Loading and Details of Composite Lintel Beams

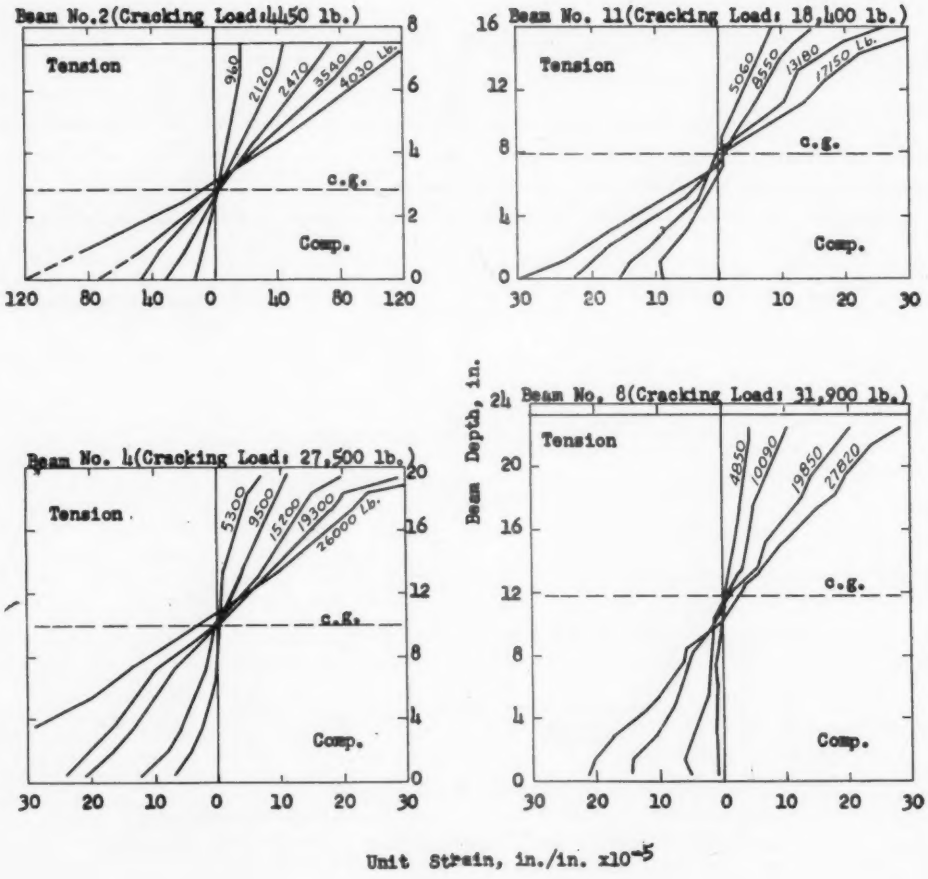


Fig. 3 Measured Strain Distribution in Beams at Various Loads

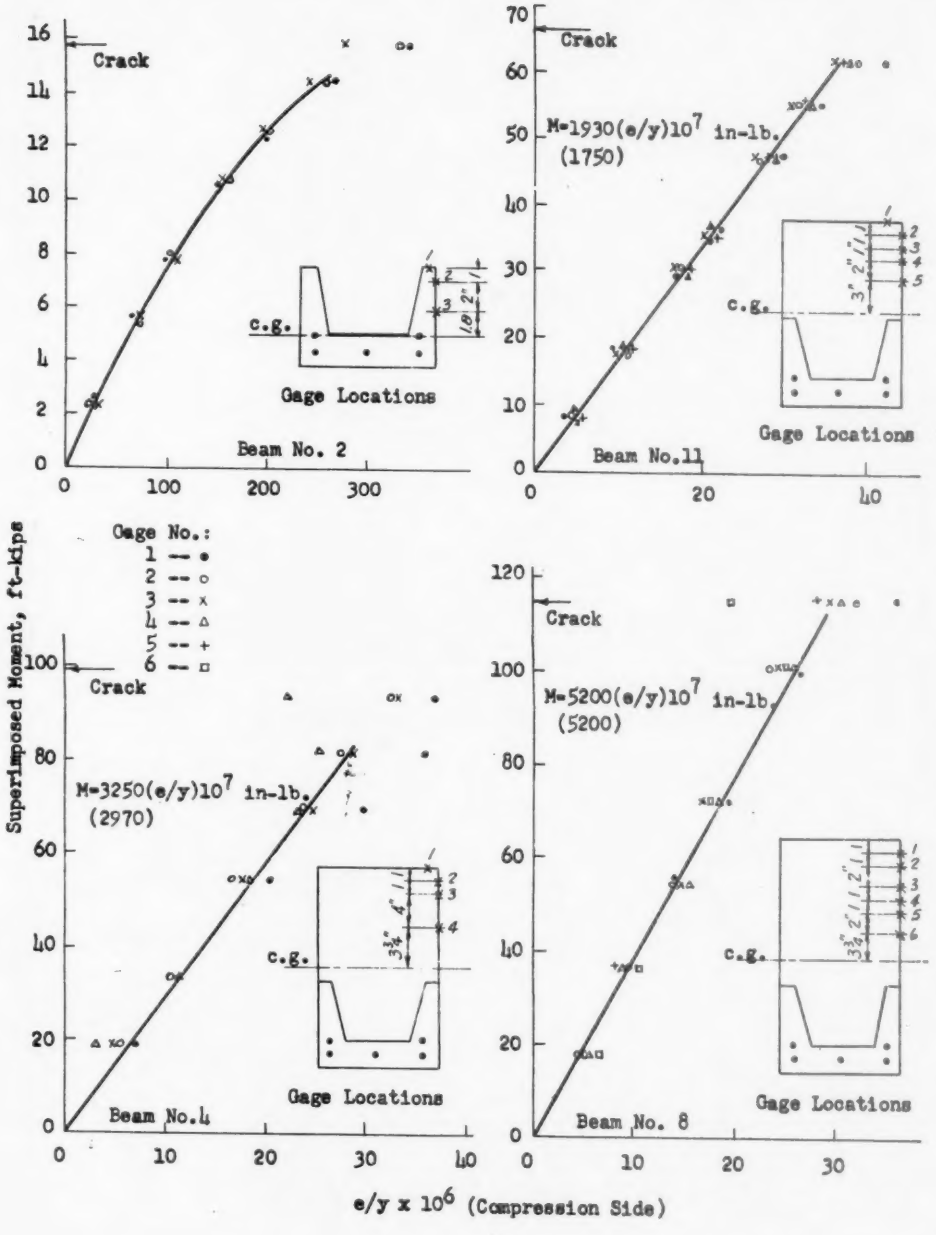


Fig. 4 Superimposed Moment vs. e/y Relationships

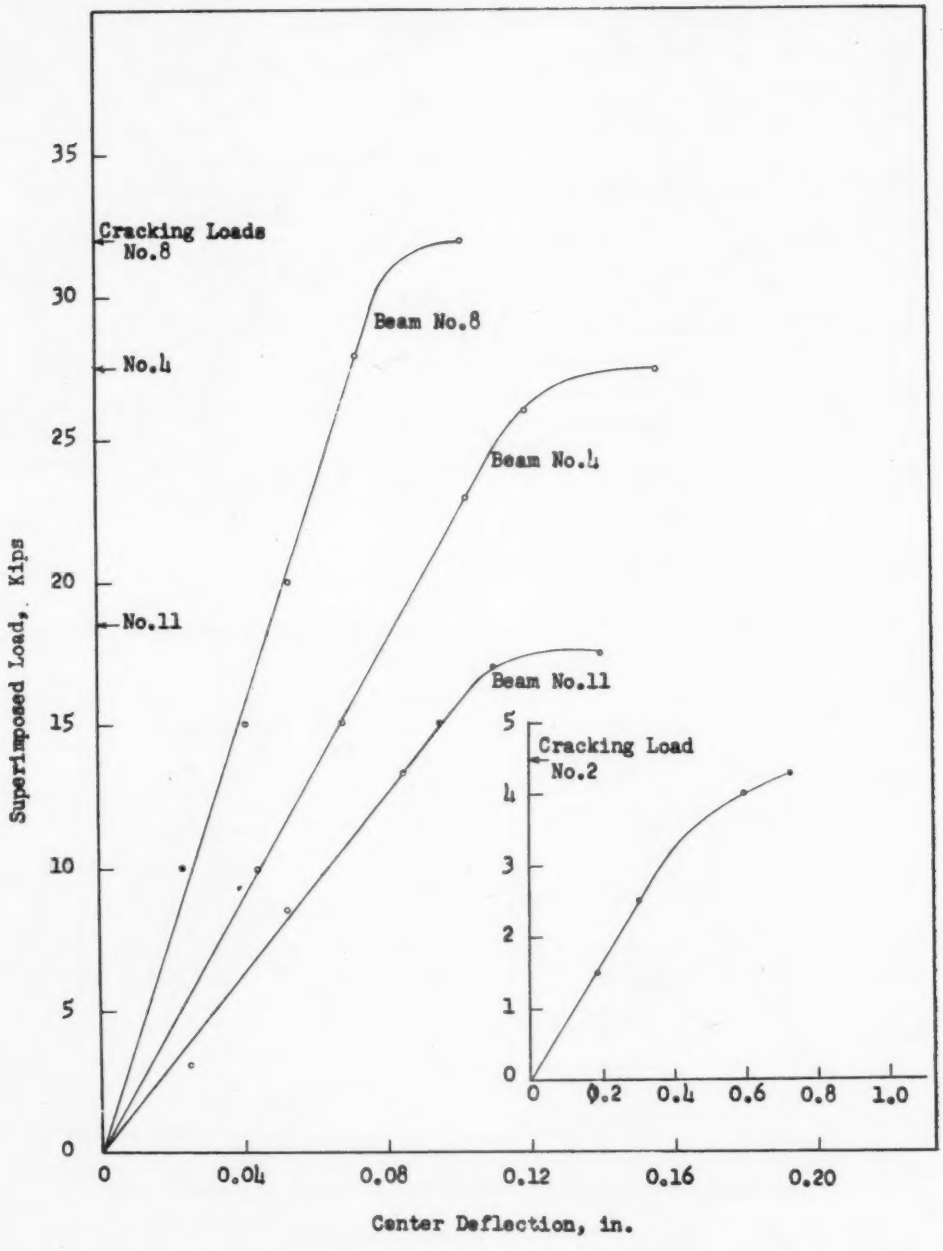
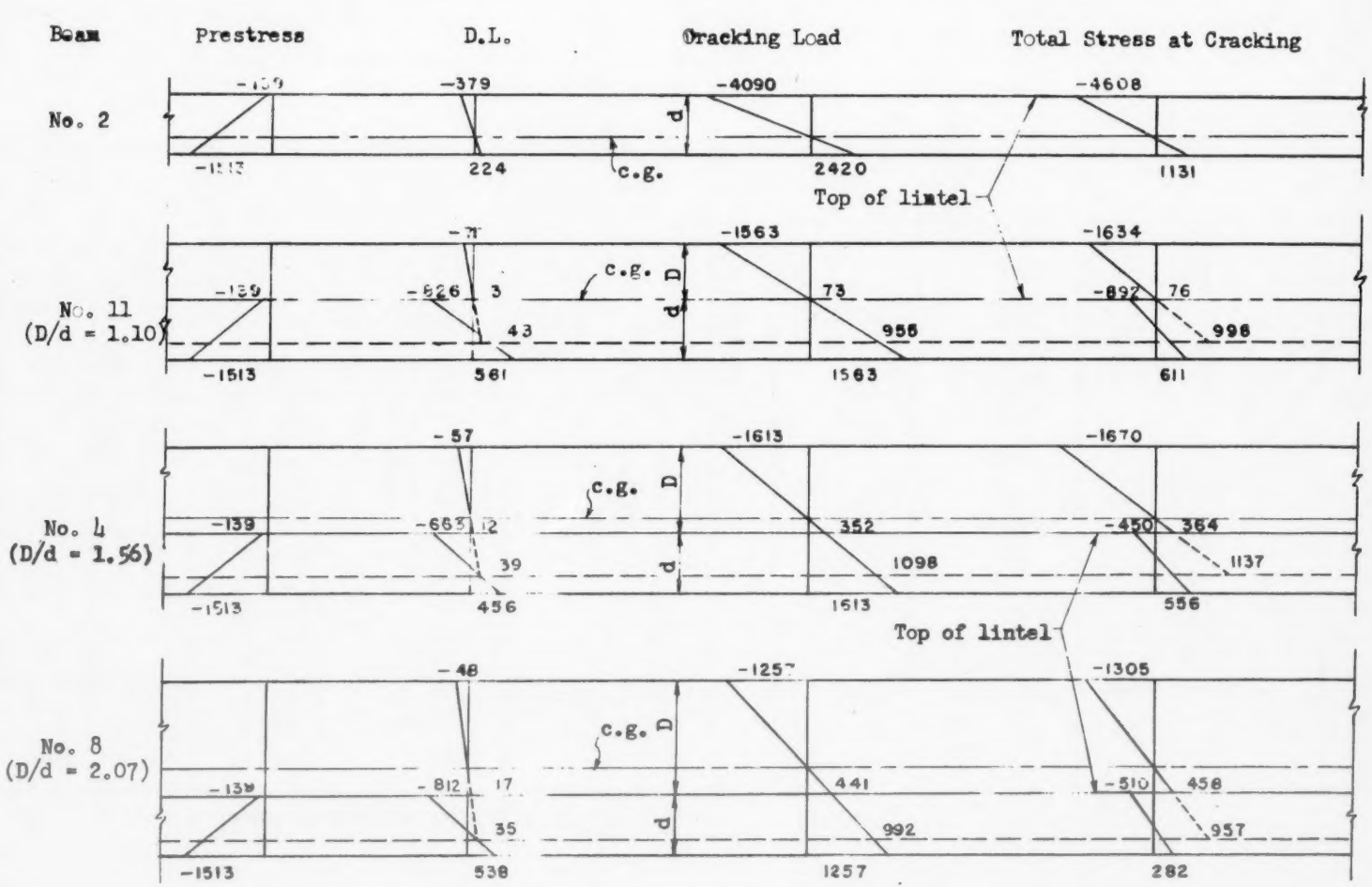


Fig.5 Superimposed Load vs. Center Deflections


Beam
Prestress
D.L.
Cracking Load
Total Stress at Cracking
No. 2
-379
-4090
-4608
224
c.g.
2420
1131
Top of lintel
No. 11
(D/d = 1.10)
-1563
-1634
-139
-826
3
c.g.
73
-892
76
43
955
998
-1513
561
1563
611
No. 4
(D/d = 1.56)
-57
-1613
-1670
-139
-663
12
c.g.
352
-450
364
39
1098
1137
-1513
456
1613
556
Top of lintel
No. 8
(D/d = 2.07)
-48
-1257
-1305
-139
-812
17
c.g.
441
-510
458
35
992
957
-1513
538
1257
282

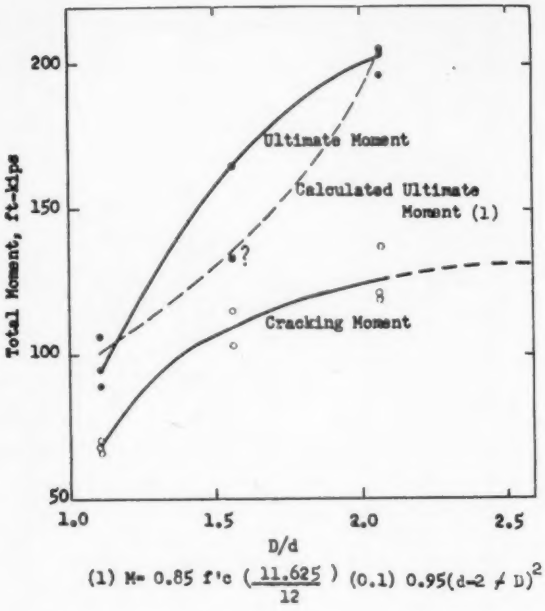


Fig.7 Total Moment vs. D/d Relationships

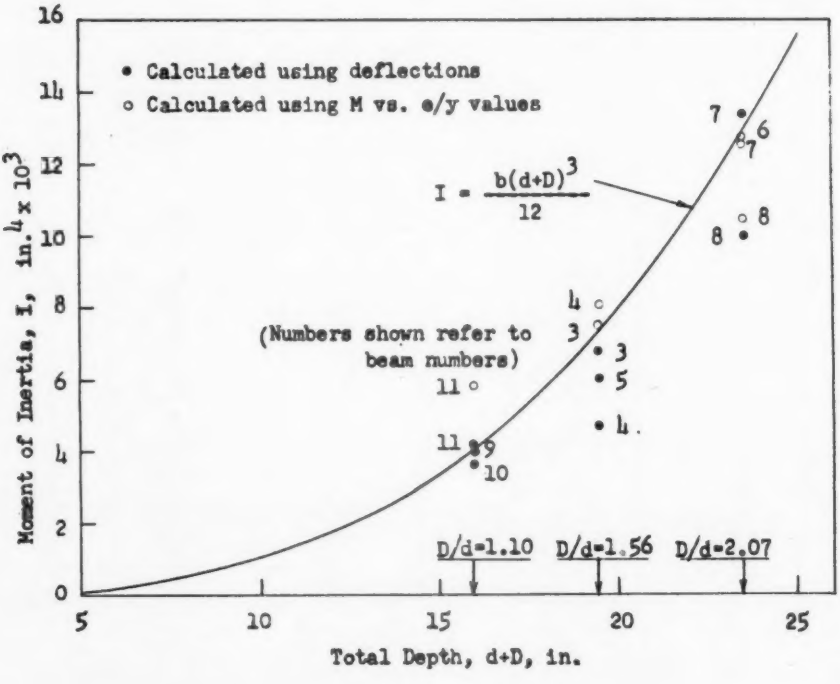


Fig. 8 Theoretical and Apparent Moment of Inertia

TABLE I SUMMARY OF TEST RESULTS OF COMPOSITE LINTEL BEAMS

BEAM NO.	D/d	CRACKING LOAD, LB.	ULTIMATE LOAD, LB.	TYPE OF FAILURE	CORE		LINTEL		AGE WHEN TESTED, DAYS	
					f'c, psi	E, psi	f'c, psi	E, psi	lintel	core
9	1.10	17,300	26,400	Shear	6520		6800		38	26
10		18,500	23,900	"	6270		6160		39	27
11		18,400	28,500	"	6560		6670		40	28
Ave.		18,050	26,300		6450 @ 32 days	3.99×10^6	6540 @ 42 days	4.28×10^6		
3	1.56	31,000	------	Shear	5590		5490		42	31
4		27,500	44,300	"	5570		5730		45	34
5		(20,000)	(34,750)	"			5990		46	35
Ave.		29,250	44,300		5580 @ 35 days	4.26×10^6	5740 @ 46 days	4.00×10^6		
6	2.07	36,600	55,300	Shear	do	do	do	do	47	36
7		31,500	54,600	"					50	39
8		31,900	52,800	"					51	40
Ave.		33,300	54,300							
1	----	4,550	5,370	Comp.	do	do	do	do	30	--
2	----	4,450	4,500	"					31	--
Ave.	----	4,500	4,935							

( ) Believed to be damaged, not included in the average.

TABLE II MOMENTS IMPOSED ON THE COMPOSITE SECTIONS

BEAM NO.	D/d (1)	TOTAL MOMENT, FT.-LB. AT CRACKING (2)	TOTAL MOMENT, FT.-LB. AT FAILURE (3)	DESIGN MOMENT (ZERO TENSION) FT.-LB. (4)	APPARENT FACTOR OF SAFETY (2) ÷ (4)
1		18,000	21,000	8,580	2.05
2		17,100	19,800		
Ave.		17,550	20,400		
9		65,500	97,100	40,600	1.71
10	1.10	70,500	88,000		
11		69,600	104,700		
Ave.		69,500	96,600		
3		115,900	--	66,800	1.64
4	1.56	103,200	165,900		
5		(76,100)	(131,100)		
Ave.		109,550	165,900		
6		137,400	206,400	89,900	1.41
7	2.07	118,600	204,100		
8		120,200	197,200		
Ave.		125,400	202,600		

( ) Believed to be damaged, not included in the average.

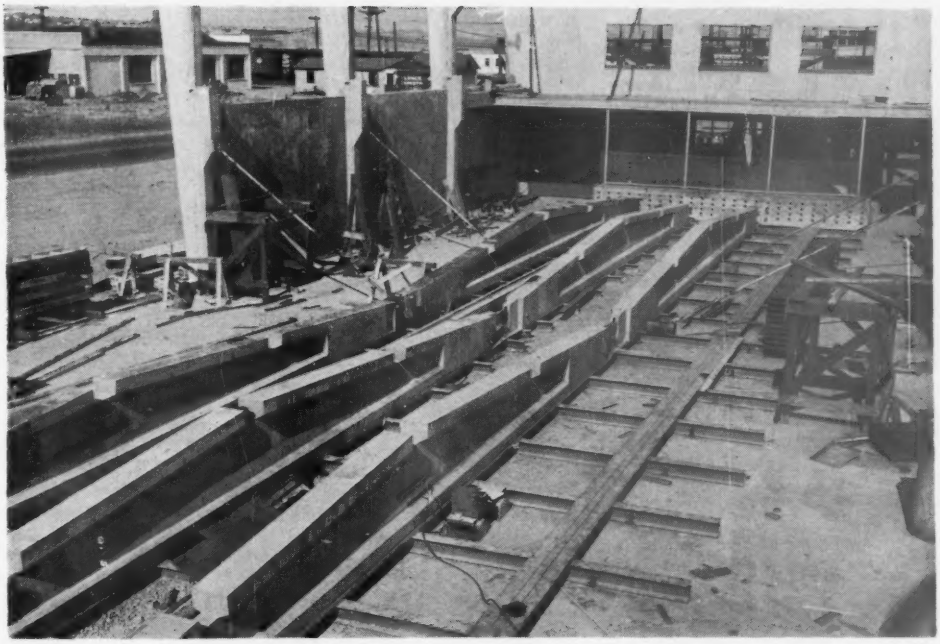


FIG. 1. Pretensioning bed and partially completed structure for plant of Concrete Engineering Company, Tacoma, Washington.

FACTORY PRODUCED CONCRETE

in the PACIFIC NORTHWEST

By ARTHUR R. ANDERSON
Concrete Engineering Company
Tacoma, Wash.

INTRODUCTION

In 1951 the Anderson Brothers founded Concrete Engineering Company, a factory for producing precast and prestressed concrete structural members in Tacoma, Washington. This factory, shown during construction in FIGURE 1, was made from its own products.

The progress made by this enterprise during the past five years has not been without considerable effort. Situated in the heart of a timber country, competition from time-honored and well-establishhed methods of design and construction have resulted in stimulating challenges to the relatively new techniques of prestressed concrete construction.

DEVELOPMENT WORK

To place the company in a strong technical position, a program of testing and development work was undertaken at the outset. The plant was equipped with a 400,000 lb. hydraulic testing machine and electric strain gage instrumentation for testing the physical properties of concrete and prestressing steel. Full scale tests on beams for bond, creep, and load-carrying capacity, up to ultimate load were carried out. As a result of this work, building codes in the Northwest have been drafted, recognizing plant-produced concrete of 7500 psi. strength with permissible working stresses up to 3000 psi.

REDUCTION TO PRACTICE

Having achieved the desired prestressing technology, attention was focused on the practical aspects of design, manufacturing and construction. Many schemes were studied and tried with a view to establishing a marketable series of standard prestressed and precast concrete products. This phase involved many facets; development of a series of standard beam sections having high structural efficiency, practicability of production of the sections using zero slump con-

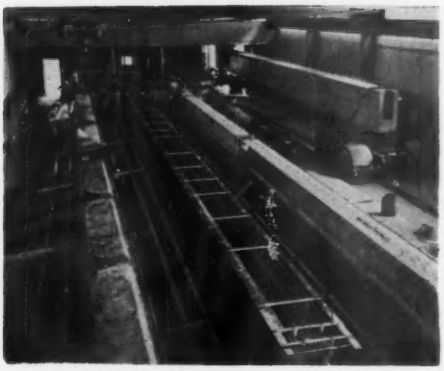


FIG. 2. Interior view of prestressing plant showing 87 ft. girder loaded on truck and steering trailer.

crete, efficient factory production methods, quality control, transportation and erection, field connection of the structural members, and æsthetic and architectural considerations.

MANUFACTURING METHODS

The plant features a pretensioning bed 105 feet long and 25 feet wide. High tensile strands are stressed initially to 200,000 psi. between the abutments, and steel forms are set up. High frequency vibrators are attached to the forms at 5-foot intervals and concrete with a W/C ratio of 3.5 gallons per sack is placed. A concrete strength of 5000 to 6000 psi. is normally obtained in one or two days, and a strength of 9000 to 11,000 psi. is obtained in 28 days. Finished products are removed from the lines by overhead bridge cranes and loaded on trucks and trailers as shown in FIGURE 2. Beams up to 100 feet long, weighing up to 60,000 lbs, can be produced and delivered from the plant.

QUALITY CONTROL

Samples from each shipment of prestressing steel are tested for tensile strength in the testing machine.

During tensioning of steel, a record of elongation is made for every five-ton load increment on the hydraulic rams for each operation.

Each day's production of concrete is tested for compression strength prior to prestress release and at subsequent ages of concrete during strength gain, by means of cube and cylinder specimens. Prior to release of prestress, every item is further checked for strength uniformity throughout its length by tests made with the Schmidt Concrete Test Hammer. Detailed shop drawings are prepared for each product manufactured, and prior to placing concrete all steel, inserts, etc., are checked.

All products are marked in accordance with an erection plan, prior to delivery.

TRANSPORT AND ERECTION

Delivery is made from the plant to the job site by truck, rail or barge. Shipments have been made up to 700 miles from the plant, ranging from Boise, Idaho, to Ketchikan, Alaska. FIGURE 3 shows a barge loaded with prestressed girders and piling for a water-front project in Ketchikan, in which 186 girders and 164 piles were used.

Concrete Engineering Company has adopted the policy of supplying the fabricated items at the Contractor's option — F.O.B. plant, F.O.B. job site, or delivered and erected.

MARKETING

Sales of prestressed concrete products is largely dependent on winning the confidence of the design engineer or architect for a project at the outset of the design stage. Promotion is therefore of a technical nature, and in most cases technical assistance and cooperation from the manufacturer are necessary. On several occasions, Concrete Engineering Company has been invited to submit alternate designs, and has won several projects on this basis. FIGURE 4 shows a large industrial building for a local public utility, which was constructed in prestressed concrete as a result of a successful alternate design.

Examples of several recent projects are shown in the following figures:

FIG. 5. National Bank of Washington during erection.

FIG. 6. Orting School Gymnasium, 82 ft. roof girders.

FIG. 7. Florida Building, near Seattle, Washington, erection of 87-foot second floor girder.

CONCLUSION

Experience at Concrete Engineering Company of Tacoma, indicates that further progress in factory produced prestressed concrete for construction can be made through the development of efficient standard structural sections, research into new structural design concepts, improvement of transportation and erection methods, and further simplification of the field connection, and the development of structures in which the elements are statically determinate for dead loads and continuous for live loads.

FIG. 3. Prestressed girders and piling loaded on barge for delivery to Alaska.

FIG. 4. Industrial Shop Building for Tacoma Public Utility. Prestressed Design for Beams and Girders Reflected Cost Saving on Alternate Design.

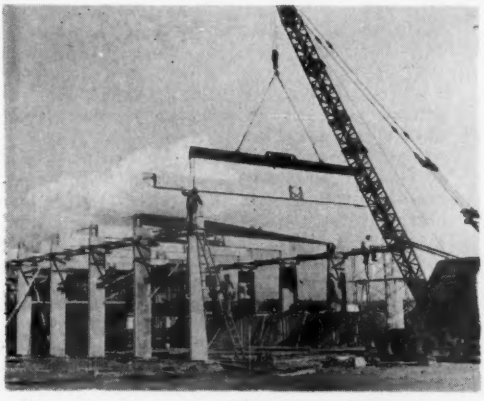


FIG. 5. Erection of Prestressed Beams on Precast Columns, National Bank of Washington.

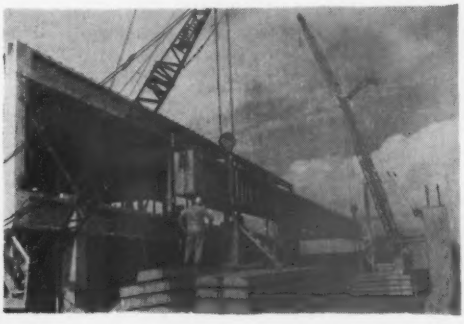


FIG. 7. Erection of 87-ft., 2nd Floor Beam, Florida Building, near Seattle, Washington. Two-story Building, 105′ x 160′ was erected in 8 Days.

FIG. 6. Gymnasium for Orting School. 82-ft. Prestressed I-Beams. Precast Roof Slabs in Foreground.

prestressed concrete now a Standard Product

By Lloyd E. Hill, *Sales Engineer*
Construction Materials Division
John A. Roebling's Sons Corp.

Recently I had occasion to read over a report dated March 16, 1945, prepared by the late C. C. Sunderland, of the Bridge Division of the Roebling Company. Its purpose was to review the past activities of our company in the field of prestressed concrete, to predict the future potential market, and also to request additional funds for research and promotional work in this field. In his report Mr. Sunderland predicted that prestressed concrete would soon become a standard building material for the fabrication and assembly of single and multiple story buildings, as well as for bridges, airport runway slabs, and highways. He also pointed out that prestressing reduces concrete sections considerably and, therefore, opens a much wider scope to the precast concrete industry. The accuracy of his predictions has been most interesting to us who have followed its development in the United States for the past eleven years.

Aside from a prestressed concrete floor slab placed in our Chicago Warehouse in 1946, it is believed that the first noteworthy prestressed concrete structure completed in the United States was the Turkey Creek Bridge in Madison County, Tennessee, in 1950. You can therefore see that for all practical purposes the history of prestressed concrete structures in this country dates back less than six years, with the greatest evolution taking place during the past three years.

The general subject of prestressed concrete is a broad one. It embraces circular and lineal structures, pre-tensioning and post-tensioning methods, as well as different methods for each. Most of them have merit depending on the conditions of applications, the majority using high strength wire of special characteristics in one form or another as a tensioning element. High strength bars serve this purpose in certain post-tensioning methods.

The theory of prestressed concrete design is quite simple to those familiar with structural steel and reinforced concrete design. Generally speaking, however, consultants have been reluctant to design in prestressed concrete. This is due largely to the fact that the field has been so new and specialized and, as a result, unfamiliar. The same thing holds true for the contractor. He can estimate conventional jobs from past experience, but when it comes to fabricating prestressed concrete members he has been in an experimental field. As a result, many consultants have deemed it advisable to make two designs, one in prestressed concrete, and the other in some more conventional method where they want to consider prestressed concrete. They justify the increased cost of making two designs by the more spirited bidding that results. It is encouraging to report that on all sizeable projects to date where prestressed concrete has been specified as an alternate to other methods, the successful bidder has chosen the prestressed

concrete alternate.

The Lake Pontchartrain Bridge is one good example of prestressed concrete economy. Here where there is large duplication of precast pre-tensioned members the precast pretensioned design won hands down over the structural steel design without considering the maintenance penalty on the steel alternate.

Port of New York Authority Pier "C" recently completed in Hoboken, New Jersey and Pier "A" now nearing completion are other examples. Supported on steel H-piling with cathodic protection the post-tensioned pile caps and precast pre-tensioned deck were an alternate to a conventionally reinforced concrete design for Pier "C". Only one of the general contractors selected the conventional reinforced design for his bid and his price was way out of line. As a result no alternate to the prestressed concrete design was considered on Pier "A".

Such other major bridges as the New Gandy Bridge in Florida, the Garden State Parkway Bridges in New Jersey, Hampton Roads in Virginia, Pelican Island in Texas, Broad River in South Carolina, and more recently The Sunshine State Parkway in Florida, and extension to the Garden State Parkway went prestressed concrete in direct competition with steel or reinforced concrete.

This experience has given designers added confidence in prestressed concrete, and as a result many jobs are now being bid with no alternate to the prestressed concrete design. In addition many county and state highway departments have adopted standard pretensioned designs for bridges and piles.

The progress in the building industry, which by the way has the greatest potential market for prestressed concrete, has followed much the same pattern only in a less spectacular way. The pioneers in this field have been the many fabricators of precast pretensioned products. One year ago these fabricators numbered approximately fifty. They have grown to approximately one hundred in number and this is not taken as a permanent situation. We predict that within a short interval of time there will be casting facilities to serve every city in the country. Schools, single and multiple story warehouses, office and factory buildings, stores, shopping centers, and stadiums are the most common type structures that have been built. Most of these structures are precast and utilize standard double-T's, channel slabs, girders, and purlins as fabricated by the pre-tensioning yards.

The use of high strength seven wire uncoated stress-relieved strand is the accepted standard for precast pre-tensioned members. Sizes range from ¼" through ½" in diameter. The 5/16", ⅜" and 7/16" sizes are in regular and continuous production and therefore usually are available on fairly short notice. Deliveries on the ¼" and ½" sizes vary with mill conditions and inventory. These strands have a minimum ultimate strength of 250,000 p.s.i., and a minimum yield as defined by the .2% offset method of .85 of the guaranteed minimum ultimate.

The characteristics of the wire used in the many post-tensioning systems have also been standardized. The most common sizes are .196", .250" and .276" diameter. Since this wire is perishable if stored for long intervals of time and demand has not reached a point to warrant regular production, deliveries of wire are necessarily not as good as for the seven wire strand.

As in most industries standardization, quality control, and factory production are the keys to maximum economy in the prestressed concrete industry. The more successful fabricators have spent considerable sums of money in plant equipment and layout. Most all use the pre-tensioning method with long line prestressing to gain maximum economy of the placing and tensioning operation, and to eliminate the necessity of relatively costly end fittings and the grouting operation. Until recently this has required straight line prestressing, which results in the same eccentricity of prestressing force throughout the length of the member. For long spans and certain loading conditions this creates critical stresses over the supports when the center of gravity of the prestressing force was located at its optimum point at the section of maximum moment. To satisfy the allowable concrete stresses at the support points the eccentricity of the tendon had to be reduced and the amount of prestressing force increased. Where conditions warrant, it is now standard practice to deflect some or all of the strands in a pre-tensioned member in the shape of a catenary that approximates the shape of the bending moment diagram, thus placing the c.g. of the tendon at its most efficient location at all points along the span.

The idea of partial prestressing, although long recognized in Europe, is just now starting to gain momentum in this country. It is well adapted to structures that are subject to static loadings, and has the advantage of lowering the concrete stresses under the dead load condition. This permits greater use of light-weight aggregate, longer spans and better camber control. It also results in less stress loss due to plastic flow in the concrete and relaxation in the steel.

Where casting facilities are close at hand it is advisable for the engineer and architect to work with the fabricator, and if possible to design around the standard sections that the fabricator has available so as to cut form and production costs to a minimum. Quite often one hears the complaint from the designer that he would gladly design in prestressed concrete if the proper facilities were available near the job-site to fabricate the members. Prospective fabricators on the other hand state that they desire to enter into the prestressed concrete field but are waiting for an initial job before starting into business. This situation is best exemplified by recent bid experience. In jobs where design engineers could foresee at most two sub-contract bids for the prestressed concrete portion of the contract, the general contractor has received as many as four bids

and all of a competitive nature.

If no standard shapes are available to the designer he must realize that economy of design will result solely from a practical approach to the problem. Steel forms are desirable but expensive. Therefore, utilizing one standard cross-section for a variety of spans may be desirable. It may be advantageous to vary the spacing of the members and/or the amount of prestressing force to further this goal. Simple cross-sections that are easy to form also add to the overall economy. Sometimes it is well to increase the amount of concrete over that theoretically required to reduce overall costs. Sketching the cross-section to scale is a big help in the final analysis of a section. Members should be detailed with adequate tolerance to insure that they will fall into place with a minimum effort during erection.

Design and construction specifications are also becoming standardized. The Bureau of Public Roads Criteria has been of much help, but now out of date in part. This is natural since much has been learned through research and production experience in this infant industry since 1953 when this "Criteria" was written.

The American Concrete Institute Recommendations for Prestressed Concrete Design and Construction were presented to the general committee for discussion at the A.C.I. meetings in Philadelphia last February after three years' effort. For the most part it was well received as witnessed by the small amount of criticism voiced by the experts attending the discussion. The revised draft is scheduled for vote by the various committee members late this fall, after which it will be printed in final form for distribution early next year.

It can readily be said that prestressed concrete is established as an important structural resource along with reinforced concrete and structural steel, and it seems obvious that the growing acceptance together with the experience gained by designers and fabricators will further promote its use and enable it to assume its proper perspective.

EXPOSED STRUCTURAL FRAMING FOR LOW COST SCHOOLS

By Ross H. Bryan, *Consulting Engineer. Nashville, Tenn.*

Fig. 1. Structural roof frame for one typical section of the Southside School. Note tie plates connecting joist and grooves for continuity steel in beams.

The need for additional classrooms in our public school systems is rapidly becoming critical in most of the larger cities throughout the United States. The rate of construction of school facilities has not been able to relieve the congestion which is present in our school systems today. Many communities have adopted the practice of building temporary or portable classrooms to give temporary relief to congested localities. In theory this practice appears to be acceptable however, it has not been successful in most cases because the temporary units soon become accepted as permanent additions to school facilities once they have been put into use. The result is the accumulation of a large number of classrooms which do not meet minimum standard requirements for space, lighting, heating, ventilation or fire safety. Their cost in many cases, when maintenance is considered, will prove to be equal to or greater than the cost of constructing a well designed, permanent, low-cost building.

It is well known by architects and engineers that the cost of the structural system of most buildings is a fairly small percentage of the finished cost of the building and therefore very little should be expected in the way of savings on overall costs by altering or experimenting with the cost of the structural framing system. The difference in cost between schools of the low cost and upper cost bracket is usually the result of differences in wall, ceiling and floor finishes and the mechanical and electrical equipment furnished.

If an economical framing system is used which, when left exposed in the ceiling, gives a presentable appearance when painted the structural frame of the building may be used for both structural and architectural expression and interior finish may be cut to a minimum through the elimination of ceilings, plaster and wainscoating.

A building of this type can be finished at any time in the future with ceilings and floors of any type when the construction budget may not be strained to the breaking point. It may be questionable whether ceiling finishes on properly designed structural members are necessary at all. It is possible to arrange the structural members so that the acoustics of the rooms may be improved by the presence of the member itself which may serve as a sound baffle. Precast slabs can be fabricated of sound absorbing materials and when painted will present an attractive and fire safe structure which will not be subject to excessive maintenance costs.

The prestressed concrete fabricator is the logical answer to the production of prefabricated structural elements for low-cost school construction. The use of his product permits the contractor to establish a firm cost for the greater portion of the cost of the structural frame. Since these members are fabricated in a plant they can be produced regardless of inclement weather or poor ground conditions. If ordered sufficiently in advance these units can be ready for use on the structure as soon as the supporting wall or columns have been completed by the contractor.

The above principles have been incorporated into the design of two school buildings by Architects Thomas, Ross and Stanfill of Jackson, Tennessee. The first structure was the Southside School at Jackson, Tennessee, covering some 70,000 sq. ft. of floor space. In this building the structural frame was so arranged that it blended into the architectural features of the building and was left painted and exposed in the completed structure.

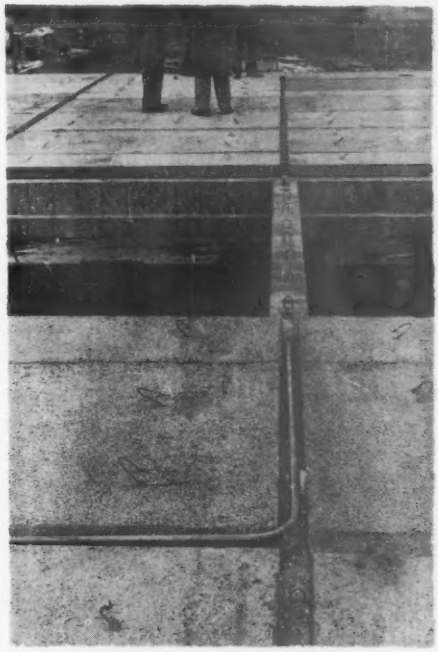


FIG. 2. Slabs bearing on joists and details of conduit in beveled edges of slabs.

The construction consisted of light weight, precast, roof slabs of insulating concrete which were 4" thick. The slabs spanned 8½' and were supported by 8x16" prestressed multi-unit joists which spanned the 24' classroom dimensions. The joists were so spaced that all classroom partitions were located underneath a framing member. The joists were supported by prestressed multi-element beams of a special design which permitted the joists to be set on a ledge of the beam and reduced the total depth of the construction to 22" from the top of the precast slab to the bottom of the corridor beams. The beams were made continuous over the columns by means of mild reinforcing steel which was placed in a groove made in the top of the beam for a calculated distance on each end. The joists were tied together over the beams by means of steel plates

FIG. 3. Prestressed channel slabs bearing on prestressed multi-element beam in shop building.

which were bolted to anchor bolts cast in the ends of each roof joist. The multi-element beams and slabs were fabricated by the Nashville Breeko Block Company and trucked 140 miles to the building site.

The slabs were fabricated in panels up to 4′ in width and the edges of the slabs were beveled so that conduit could be placed in the grooves formed by the bevels of adjacent slabs. After the conduit had been placed the grooves were filled with Vermiculite concrete. The lighting fixtures were attached to junction boxes through openings cut in the slab in the field. Roofing was applied directly to the top of the slabs.

Construction similar to the above was used in a Gymnasium at Milan, Tennessee, in which 100′ girder spans were required. The girders were 5′6″ in depth and were pretensioned with 54-⅝″ deflected strands. They were fabricated in the plant of T. L. Herbert & Sons Company, Nashville, Tennessee, and shipped by rail to the construction site. The girders supported 8x12″ multi-element joists which were spaced 5′ on centers and supported 4″ precast, light weight, insulating, structural slabs. These slabs were beveled on the edges to receive the lighting conduit. The panels were made in widths up to 4′. The purlins were fastened to the prestressed girders by means of clip angles and bolts cast into the member. The slabs were anchored to the tops of the purlins by means of wire ties extending from the top of the purlin into the keyed space between slabs which was then filled with concrete. It is important to realize that the precast slabs in this case served three building purposes; a structural slab, an insulating unit and formed a raceway for the electrical conduit. The appearance of the slab when painted is acceptable, it is clean as well as fire proof. The nature of the material and the texture of the finished slab provide some acoustical treatment. The presence of the beams also provide acoustical treatment. It has been found that the acoustics of this type of construction are acceptable for the use to which they are being put.

FIG. 4. The 100 ft. prestressed girders of the Milan Gym and the multi-element beams supporting the precast slabs.

DUOFLEX CASING for PRESTRESSED CONCRETE INDUSTRY

By Lewis E. Weeks, *President*
Flexico Products, Inc.

As a result of the past few year's experience with the rapidly growing Prestressed Concrete Industry, certain facts and conclusions have come to light.

This new industry has been striving to develop and establish a reputation based on it's record and experience in the field of heavy construction. In order to do so, it has been necessary to demonstrate improvement over previously established methods of construction and at the same time prove itself more economical.

Until recently the requirement for grout casing has been partially satisfied by the use of standard flexible metal hose products designed for other purposes. Demands for large quantities of casing at a cost low enough to enable contractors to compete against the older established methods, caused metal hose manufacturers to classify this casing as a "shop fill" product. This material was to be manufactured during slow periods and sold at a price designed to barely cover the costs of material, labor, overhead and shipping expense. Even with casing classified as "shop fill", the contractors and proponents of post-stressed concrete methods, felt the need for a less expensive casing. The manufacturers, always willing to please, were left with only one alternative—to decrease the cost of base materials by cutting down on the metal thickness.

The results of this course of action were soon evident. Troubles developed that had a tendency to offset the savings. One case is that of a large storage tank being constructed in the West, where a few cables were broken from over-tension. Efforts were made to replace the cables, but it was found that the casing had crumpled at the time of the break and had blocked the passageway for the new cable. Damage in shipment caused by lack of crush strength was another result of decreasing the wall thickness. In covering long lengths of cable or rod, the casing will tend to part. This is due to the weight of the cable or rod combined with the friction against the inside of the casing.

About six months before the first annual meeting of the Pre-Stressed Concrete Institute in 1955, Flexico Products, Inc., in conjunction with John A. Roebling's Sons and Stressteel Corporation saw the need for a casing specially designed for the prestressed concrete industry. Flexico undertook the task of creating a new, lighter weight casing, without sacrificing the all important crush strength. "DUOFLEX" casing was developed and tests were made with assistance of American Smelting & Refining Research Laboratories. In spite of increasing prices of steel, labor and manufacturing costs, the actual delivered cost to the contractor has decreased approximately 50 percent.

During the past 12 months, hundreds of thousands of feet of "DUOFLEX" casing have been shipped from coast to coast in wire bound coils, unboxed, with no evidence of damage in transit. Less rugged material would have required costly protective boxing or created a problem in freight damage claims.

The writer attended the first annual meeting of the Prestressed Concrete Institute to study as much as possible about the problems confronting the contractors, using casing for the post-tension method. At the time of this meeting, the prospects for the extensive use of the post-tension method were not too encouraging. All emphasis seemed to be placed on the elimination of post-tensioning wherever possible. In spite of this disheartening view, developments were continued.

It was found that a simplified method of attaching grout fittings was needed to eliminate costly field work. Flexico is now furnishing, at the contractor's request, a built-in grout fitting to be used with a ⅜" pipe nipple wherever required in a length of casing. This has eliminated the purchase of special saddles, and no brazing or wrapping time is necessary in the field when preparing the casing for the grouting operation.

As is the case with most materials, DUOFLEX must be varied to fit special requirements. When casing is to be shipped, handled and installed with wire or cable already inserted, it is best to increase the metal thickness to add crush strength and scuff resistance.

Since the assemblies are heavy enough to require their handling by cranes and slings, there is a factor of extra rough handling and excessive weight exerted on small areas. While increased metal thickness increases the cost of the casing, shipping, charges become less as the casing is shipped at the rate of steel cable or wire. The light weight of casing as compared to its bulk has caused the railroad and trucking companies to set a high transportation rate on material of this type.

DUOFLEX casing is also furnished in galvanized steel of slightly heavier weight for special application, such as pier construction, when used in and over salt water. While it is generally agreed that rust accumulation resulting from long storage on the job site in normal weather conditions, has no serious effect on casing; however, an over-exposure to salt water, may cause enough corrosion to weaken its total structure. DUOFLEX casing is now available in continuous lengths up to 120 feet. Lengths up to 400 feet can be furnished by means of splices that are brazed and therefore, are as strong as the rest of the length.

In the last 12 months since the convention at Fort Lauderdale, it has been evident that the use of the post-tensioning method was underestimated. There have been many contracts let throughout the country, where the post-tension method has been found most adaptable. The volume of casing for post-tension work has been far in excess of that predicted at the convention. We now believe that post-tension must go hand in hand with pre-tension as one of the tools of the industry—a tool which opens the door to jobs beyond the range of pretensioning beds.

MEMBERSHIP QUALIFICATIONS

SCHEDULE OF DUES

Anyone engaged in the production of prestressed concrete, or in the production of materials or equipment allied to the prestressing industry; licensed architects and engineers; engineers and architects in training, and students enrolled in accredited schools of architecture and engineering, are invited to apply for membership in the Prestressed Concrete Institute. Membership classifications have been established as follows:

ACTIVE
Organizations and individuals actively engaged in the production of prestressed concrete products.
Annual Membership Dues — **$250.00**

ASSOCIATE
Organizations and individuals engaged in the production of materials and equipment allied to the prestressed concrete industry.
Annual Membership Dues — **$250.00**

PROFESSIONAL
Limited to registered architects and engineers.
Annual Membership Dues — **$ 25.00**

JUNIOR
Limited to architects and engineers in training.
Annual Membership Dues — **$ 15.00**

STUDENT
Limited to students of accredited architectural and engineering schools.
Annual Membership Dues — **$ 5.00**

APPLICATION FOR MEMBERSHIP IN PRESTRESSED CONCRETE INSTITUTE
(A non-profit corporation)

1. Name and address of company applying for membership:

_____

_____

_____

2. Name of person to represent company in Institute affairs:

_____

3. Type of membership applied for:

☐ Active (manufacturer)

I have done the following types of prestressing:

- ☐ Post-tensioning
- ☐ Pre-tensioning
- ☐ Combination of both of above
- ☐ Precasting
- ☐ On-the-job post-tensioning
- ☐ On-the-job pre-tensioning
- ☐ Design

☐ Associate (related business)

I am in the business of_____

_____

_____

☐ Professional (registered architect or engineer)
I hold certificate No._____of the State of_____.

☐ Junior (limited to architects and engineers in training)

☐ Student (limited to students of accredited schools of architecture and engineering)
Name of School_____

I understand the financial arrangements and am not to make any payment until approval of this application for membership.

Signed:_____

By_____

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